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ABBREVIATIONS.—P=Paper; D=Discussion; I=Illustrated.
Names of authors of papers, etc., are printed in *italics*.

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WORK OF THE HYDROGRAPHIC BRANCH OF THE UNITED STATES GEOLOGICAL SURVEY IN NEW ENGLAND AND A DISCUSSION OF THE METHODS USED FOR ESTIMATING STREAM FLOW.

BY H. K. BARROWS, MEMBER OF THE BOSTON SOCIETY OF CIVIL ENGINEERS.

[Read before the Society April 19, 1905.]

Origin and Development of the United States Geological Survey. — The United States Geological Survey was created by an act of Congress approved March 3, 1879. It is one of the bureaus of the Department of the Interior, among other branches of this department of governmental service being the Patent office, Pension office and Bureau of Indian affairs. Prior to 1879 there had been five Federal surveys engaged in mapping portions of territory of the United States. The oldest of these — the Coast and Geodetic Survey — had restricted its survey to the coast line, but had extended its geodetic and scientific investigations over various parts of the country. The other four organizations had made surveys and explorations in the territory west of the one hundredth meridian. When the United States Geological Survey was created, all these earlier surveys were discontinued except the Coast and Geodetic Survey, which, as a part of the new Department of Commerce and Labor, continued its work on the lines which have been followed for more than half a century.

The directors of the Geological Survey have been Mr. Clarence King, 1879–1881, and Major J. W. Powell, 1881–1894, these two men having been prominent in the surveys and investigations made before 1879. The present director who succeeded Major Powell is Mr. Charles D. Walcott.

The original intention was for the Geological Survey to make "a classification of public lands, and the examination of the geological structure, mineral resources and products of the national domain." A good topographic map was at once recognized as a necessity in all of these investigations, and the topographic sheets of the Geological Survey, as published, have proven of very great value and use in the different parts of the country.

The examination of the amount and quality of water supplies, the study and mapping of forest reserves, have also been carried out under a broad interpretation of this original purpose.

The growth of the work of the United States Geological Survey is perhaps well shown by the increase of its annual appropriations. The first appropriation in 1879 was in the neighborhood of $106 000. By 1900 the total appropriation had reached the sum of about $845 000. The last few years has seen a rapid increase, — the fiscal year of 1904–5 there being appropriated a total amount of about $1 505 700; the larger items being as follows:

Topography	$300 000
Geology..	150 000
Alaska	60 000
Hydrography	200 000
Chemical and physical researches..................	20 000
Mineral resources	50 000
Engraving and printing maps	100 000
Surveying forest reserves	130 000

It is also interesting to note the progress on the topographic sheets of the country. During the first year, 1879–80, some 3 400 square miles were mapped on a scale of about four miles to the inch. During 1902–3 some 31 000 square miles were covered on a larger scale of one and two miles to the inch.

Present Organization of the Survey. — As shown by the following table, there are five primary branches; and in the twenty-five years since 1879 the number on the permanent force of the Survey has grown from 39 to 738. This number is, of course, temporarily increased during the field season by the employment of assistants and aids in surveying, and by laborers. So that during the field season of 1903 there was a total of 1 200 employees.

ORGANIZATION OF UNITED STATES GEOLOGICAL SURVEY.

Administrative	Executive
	Disbursements and accounts
	Library

- Geologic
  - Geology and paleontology
  - Alaskan mineral resources
  - Mining and mineral resources
  - Chemical and physical researches
- Topographic
  - Topography
  - Geography and forestry
- Hydrographic
  - Hydrography
    - *Districts:* (1) New England, (2) New York and Great Lakes, (3) Middle Atlantic States, (4) Southern States, (5) Central States, (6) Texas, (7) Western States with numerous subdivisions.
  - Hydrology
  - Hydro-economics
  - Reclamation service
- Publication
  - Editorial
  - Engraving and printing
  - Documents

The director of the Survey is appointed by the President of the United States; all other permanent positions are filled by means of the Civil Service Commission by competitive examination.

Hydrographic Branch. — Hydrographic investigations by the Geological Survey began as a distinct feature of this work in 1888, under the charge of Mr. F. H. Newell, the present chief of this branch. The first specific appropriation for gaging streams was not made, however, until 1894; at that time $12 500 being set aside for this purpose by Congress. This amount has been increased from time to time, and during the fiscal year of 1904–5 it was $200 000.

What was first known as the Division of Hydrography has been superseded by the Hydrographic Branch, which includes the branches of hydrography, hydrology, hydro-economics and the reclamation service. The stream measurement work of the Hydrographic Branch had originally been in connection with irrigation projects and investigations in the West, but has gradually extended to cover a considerable portion of the East as well. When, in 1902, the National Reclamation Act became a law and the work of improving the vast arid areas began to be systematically undertaken by the Reclamation Service in the West, the stream measurement and irrigation work were separated, although the general administration of both divisions is the same, and much work is carried on in coöperation.

The purpose of this paper is to discuss hydrography in the eastern part of the country, and especially in New England.

During the earlier years of stream measurement here it was impossible with the appropriations on hand to employ any considerable permanent force, and the work was very largely done by securing inexpensive or gratuitous coöperation from men who were employed in some permanent occupation, so that they were not dependent on remuneration from the stream gaging work. Thus in many cases measurement of flow was made by professors of geology or civil engineering in connection with educational institutions, or by local civil engineers.

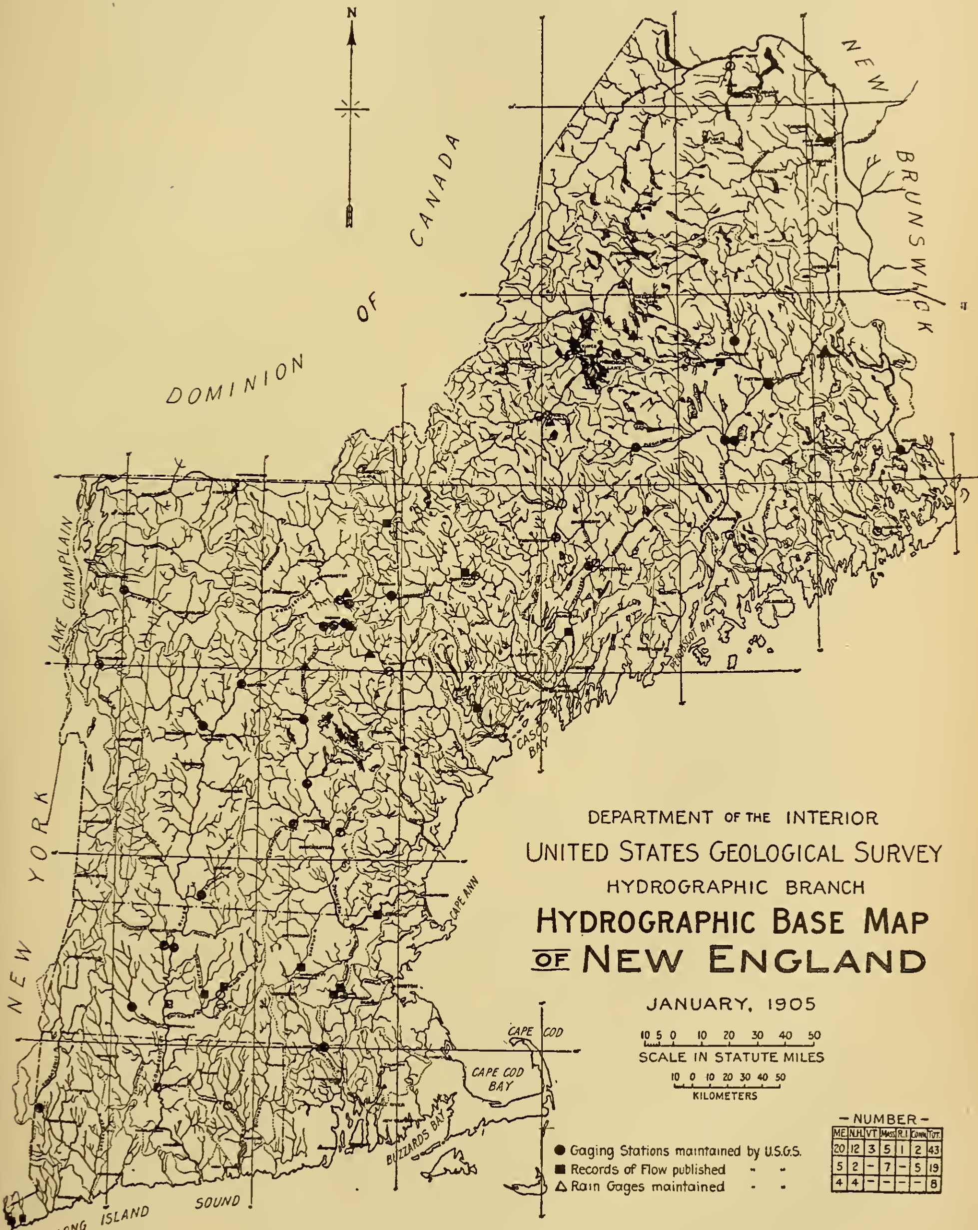
As appropriations increased, however, it became possible to reorganize the work on a more permanent basis, and to employ salaried hydrographers to carry on systematic measurements of flow. The states and territories have been grouped into hydrographic districts, each under a district hydrographer, who is responsible for the work done in his section. Thus, east of the Reclamation Service there are six of these districts, namely: (1) New England, with headquarters at Boston; (2) New York and the Great Lakes district, with headquarters at Utica; (3) the Middle Atlantic States, with headquarters at Washington; (4) Southern States, with headquarters at Atlanta; (5) Central states, with headquarters at Chicago; (6) state of Texas, with headquarters at Austin.

At the present time measurements have been or are being made on most of the principal rivers in the United States, and further investigations are constantly being made on other hydrographic matters.

Hydrographic Work in New England. (See Plate I.) — The first current meter stations of the Geological Survey were established in the fall of 1900 at Orford, N. H., on the Connecticut River, and Gaylordsville, Conn., on the Housatonic River. Although there have been in several cases estimates made of flow at points on some of the more important water-power streams by private parties from time to time, the real work of measuring stream flow in a systematic and general way began in the state of Maine in the summer of 1901.

In this state the rivers and attendant water powers, which are among the finest in the world, had long been recognized as of the greatest importance in the development of its industries. About this time the demand for more information of a definite nature regarding the quantity of water available at different times of the year culminated in an offer of coöperation in field work coming from several mill owners, lumbermen and interested parties in the Kennebec valley. They raised a sum

PLATE I.


N
DOMINION OF CANADA
NEW BRUNSWICK
NEW YORK
LAKE CHAMPLAIN
PENOBSCOT BAY
CASCO BAY
CAPE ANN
CAPE COD
CAPE COD BAY
BUZZARDS BAY
LONG ISLAND SOUND
DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
HYDROGRAPHIC BRANCH
HYDROGRAPHIC BASE MAP OF NEW ENGLAND
JANUARY, 1905
10 5 0 10 20 30 40 50
SCALE IN STATUTE MILES
10 0 10 20 30 40 50
KILOMETERS
● Gaging Stations maintained by U.S.G.S.
■ Records of Flow published
△ Rain Gages maintained
-NUMBER-
ME. N.H. VT. MASS. R.I. CONN. TOT.
20 12 3 5 1 2 43
5 2 - 7 - 5 19
4 4 - - - - 8

of money and requested Mr. Newell, chief engineer of the Hydrographic Branch, to make a study of the Kennebec River and some of its principal tributaries. This started the work in Maine. Several gaging stations were established that year, and in the course of the next season the state began to recognize the value of the work, so that in 1903 a definite appropriation for hydrographic investigations was made by the legislature.

The work has extended from time to time, so that at present there are some twenty stations in Maine maintained by the Geological Survey and covering most of the principal rivers of the state. Following is a list of these:

River and Locality	Date of Establishing Station.
Fish River at Wallagrass	July 29, 1903
Aroostook River at Fort Fairfield	July 31, 1903
St. Croix River at Sprague's Falls, Baring	Dec. 4, 1902
Machias River at Whitneyville	Oct. 17, 1903
Penobscot River at West Enfield	Nov. 5, 1901
East Branch of the Penobscot River at Grindstone	Oct. 23, 1902
Mattawamkeag River at Mattawamkeag	Aug. 26, 1902
Piscataquis River at Foxcroft	Aug. 17, 1902
Cold Stream at Enfield	June 14, 1904
Phillips Lake and outlets in Dedham and Holden	July 7 and 19, 1904
Kennebec River at the Forks	Sept. 28, 1901
Kennebec River at North Anson	Oct. 18, 1901
Moose River, near Rockwood	Sept. 7, 1902
Roach River at Roach River	Nov. 10, 1901
Dead River, near the Forks	Sept. 29, 1901
Carrabassett River at North Anson	Oct. 19, 1901
Sandy River, near Madison	March 23, 1904
Messalonskee River at Waterville	June 18, 1903
Androscoggin River at Dixfield	Aug. 22, 1902

The work of stream measurements in other New England states began in the year of 1903, and was confined at first wholly to New Hampshire and Vermont. In New Hampshire greater impetus was given to it on account of the investigations which were being made at that time by the New Hampshire State Forestry Commission, particularly for the purpose of mapping and procuring data on the forests of the White Mountain region. They deemed it advisable to make an effort to ascertain, if possible, something of the effect of deforestation on the amount and distribution of run-off, and, coöperating with the United States Geological Survey, some seven gaging stations were established on White Mountain streams, and systematic measurements began. These have continued to the present time,

but with not, of course, sufficient data as yet to enable any definite conclusions to be drawn in regard to this interesting question.

Following is a list of the gaging stations in New Hampshire and Vermont:

River and Locality.	Date of Establishing Station.
Connecticut River at Orford, N. H.	Aug. 6, 1900
Androscoggin River at Shelburne, N. H.	May 30, 1903
Saco River at Center Conway, N. H.	Aug. 26, 1903
Merrimac River at Franklin Junction, N. H.	July 8, 1903
Pemigewasset River at Plymouth, N. H.	Sept. 5, 1903
Contoocook River at West Hopkinton, N. H.	July 9, 1903
Suncook River at East Pembroke, N. H.	Nov. 3, 1904
Israel River (above So. Branch), Jefferson Highlands, N. H.	Sept. 2, 1903
Israel River (below So. Branch), Jefferson Highlands, N. H.	Sept. 2, 1903
Ammonoosuc River at Bretton Woods, N. H.	Aug. 28, 1903
Zealand River at Twin Mountain, N. H.	Aug. 29, 1903
Little River at Twin Mountain, N. H.	Jan. 21, 1904
Ashuelot River at Winchester, N. H.	July 10, 1903
White River at Sharon, Vt.	June 30, 1903
Otter Creek at Middlebury, Vt.	April 1, 1903
Winooski River at Richmond, Vt.	June 25, 1903

The work of stream measurements in the lower New England states has only been begun. The following list shows stations which were in operation on January 1, 1905:

River and Locality.	Date of Establishing Station.
Connecticut River at Sunderland, Mass.	March 31, 1904
Deerfield River at Deerfield, Mass.	March 30, 1904
Ware River, near Ware, Mass.	Sept. 15, 1904
Quaboag River, near West Warren, Mass.	Oct. 25, 1904
Westfield River at Russell, Mass.	April 1, 1904
Blackstone River at Woonsocket, R. I.	April 5, 1904
Shetucket River at Willimantic, Conn.	April 4, 1904

This list does not include a station at Gaylordsville, Conn., established in 1900 in connection with the investigations for an increased water supply for "greater New York"; also stations on Miamus and Byram rivers in the extreme southwest of the state, established in 1903. These are operated from the Utica, N. Y., district office.

In addition to the procuring of estimates of stream flow, based on the use of the current meter, the Geological Survey is enabled to obtain through various private parties and corporations a considerable amount of data on flow, which is incorporated in the Annual Progress report of stream measurements.

The following list shows about what was obtained in that way during the year 1904:

Penobscot River at Millinocket, Me.
Kennebec River at Waterville, Me.
Cobbossecontee River at Gardiner, Me.
Androscoggin River at Rumford Falls, Me.
Presumpscot River at Sebago Lake, Me.
Androscoggin River at Errol Dam, N. H.
Androscoggin River at Gorham, N. H.
Merrimac River at Lawrence, Mass.
Merrimac River at Garvins Falls, N. H.
Sudbury River at Framingham, Mass.
Lake Cochichuate at Cochituate, Mass.
Nashua River at Clinton, Mass.
Ware River at Gilbertville, Mass.
Swift River at West Ware, Mass.
Quaboag River at West Warren, Mass.
Connecticut River at Hartford, Conn.

Distribution of Work at Present. — Beside the matter of obtaining estimates of flow of rivers, the Hydrographic Branch is devoting some time to the procuring of river profiles and such data as will be of value in pointing out to capitalists suitable sites for power development. Thus, during the past field season, a plan and profile have been made of the Kennebec River in Maine, from Skowhegan to Moosehead Lake, and similarly the Penobscot River has been covered from tidewater at Bangor to Twin Lake at Norcross. The results of this survey are a plan on which contours are given for every foot difference of elevation of water surface, while the banks of the river are contoured with a vertical interval of twenty feet. The idea is to show the fall between any two given points in detail and to show enough of the banks of the river and conditions generally at any point to indicate whether or not power developments can be considered. These plans and profiles are to be published on the same size of sheet as are the topographic maps.

It is intended from time to time to obtain general data and information regarding the rivers of New England and sites for water powers, and to publish such information and make it generally available.

A very probable field for future work of the Survey will be that of investigating storage possibilities and suggestions for making the flow of the important rivers more uniform and better sustained during the dry season.

Another subject which is about to be taken up and studied is that of evaporation; there being a very marked demand for

some actual experiments on this important feature as regards storage, especially in the northern parts of New England.

Methods used for Estimating Stream Flow. — The principal method in use by the Survey is that based upon the use of the current meter, and this discussion will be largely confined to that. It will, perhaps, be enough to say that in many cases estimates of flow are obtained by means of weirs, or dams used as weirs, and made by computation and the application of a coefficient derived from such data as are available, either from the comparatively few experiments that have been made on this subject, or by using the current meter at some point near the dam and actually measuring the flow in that way, and thus working backward arriving at a suitable coefficient.

One of the regular stations of the United States Geological Survey is at the dam of the Madison Electric Works on Sandy River near Madison, Me. This is a substantial structure of stone masonry with a level and even crest, and makes a fairly satisfactory means for measurement in this way, the principal objection to it being its considerable length, — such that under low-water conditions the flow over the dam is a very slight one as regards depth. This plant is used only during the night time, so that a gage reading taken late in the afternoon gives a very good idea of the average flow for the day.

Another station of the Survey is at the dam of John T. F. MacDonnell at West Warren, Mass., on the Quaboag River. This is a timber crib structure with crest in excellent condition, and enables close estimates of flow to be made. In this case, also, the power plant is used only during half of the twenty-four hours, so that a gage reading taken early in the morning gives under most conditions a good index of the flow for the day. In low-water seasons, however, when the effect of pondage above this point becomes very marked, it is expected to maintain an automatic gage at this point.

The matter of flow over weirs and dams is a broad subject by itself, and will not be considered further at this time.

Development of Current Meter Gaging Stations. — The methods in use are a result of the conditions under which stream measurements are made. It is essential that observations should be extended over a considerable period of time, and not only the total flow of the stream be obtained, but also its distribution, so that the amount of water available for any one day may be known. Moreover observations made with a fair degree of accuracy, covering a long time, are of greater value than more

accurate observations covering short periods. It was evident upon studying the problem that the two variable quantities must be measured and recorded from time to time: (1) The mean daily height of the river, referred to some properly fixed datum, and (2) the discharge corresponding to any given height. It was then assumed that for any given stage there would always be a certain corresponding discharge. This is not strictly true, as is well known, but for most cases it is substantially so.

Selection of Gaging Stations. — Gaging stations may be of two classes, — temporary or permanent. The permanent stations are intended to be maintained over a long series of years, and are located only after a careful reconnoissance with the idea of obtaining the very best conditions. The purpose of stream measurements varies, of course, with the part of country under consideration. Thus in Maine, or other states where the importance of rivers is largely due to their use for water power, the data regarding low and ordinary stages are the most valuable; and in locating a permanent station on such a river, conditions should be, if possible, most favorable at low and medium stages. Where storage projects are under consideration, or investigations are to be made of floods, the amount and distribution of high-water flow is important, and high-water conditions would have more weight in the location of a station.

Favorable Conditions for Current Meter Gaging Stations. — The channel at a gaging station should be, of course, as nearly straight as possible for some distance above and below the station. The bed should be permanent and regular in shape. In general the flow should be uniform and well distributed in the cross section of the stream, and it is undesirable that the velocity should, at low stages, be less than one-half foot per second in any considerable portion of the cross section. Any influence of back water from high water in streams below, or from dams, is fatal to the correct estimation of discharge. The banks should be fairly high and not liable to overflow, except during extreme freshets. A reliable party for a gage reader must live within reasonably easy access of the station. It is, of course, desirable, if possible, to have a station easily reached by rail or livery, but this should be a secondary consideration when it comes to a matter of procuring reliable records.

Classification and Equipment of Gaging Stations. — Current meter gaging stations are in most cases at a bridge, as this furnishes a convenient means for installing the gage, and for making current meter measurements. The essentials at such a

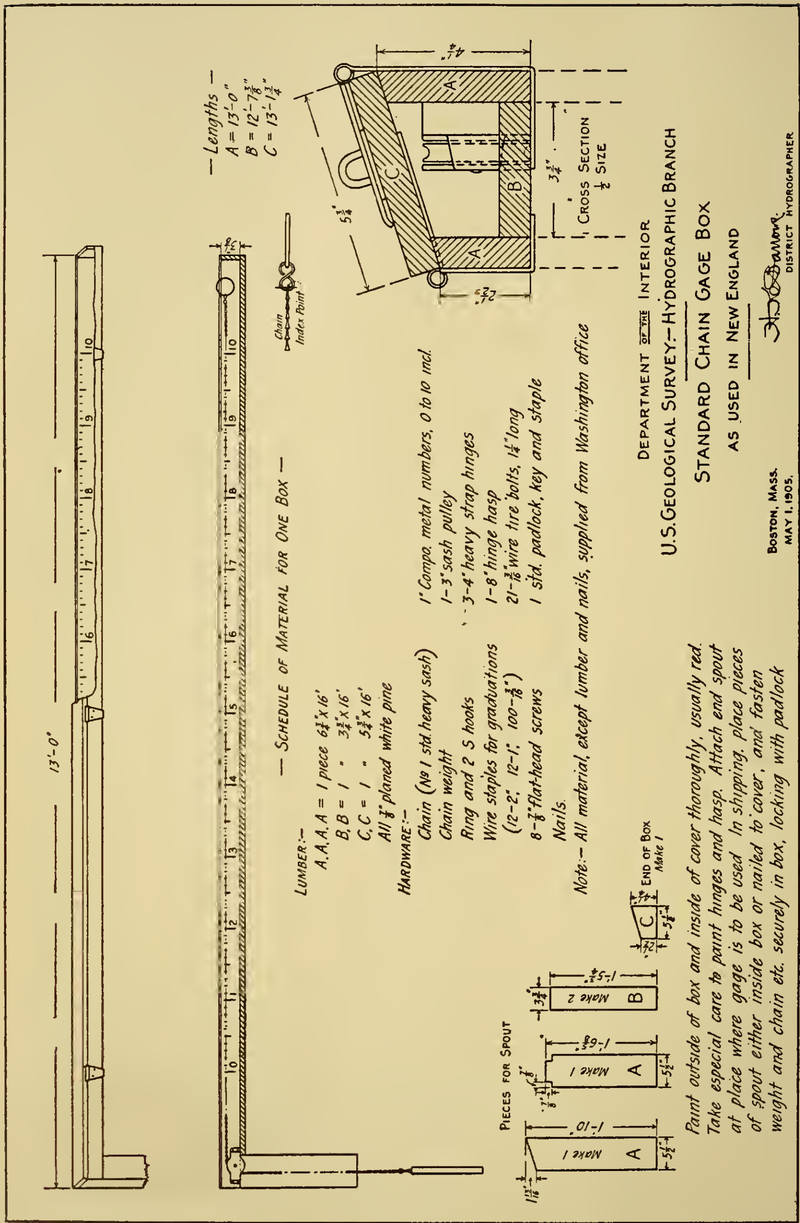
station are, a line at which soundings are made, usually from the upstream or downstream side of the bridge, which is marked off in some fairly permanent way into 5-, 10-, or 20-foot spaces, using for the initial point some permanent feature of the bridge, as either abutment; a gage for reading the stage of the water, and bench marks to fix the elevation of the zero of the gage, one of these being placed independently of the gage in such a way that check measurements can be made to the water surface by means of a steel tape, and any inaccuracy of the gage noted.

Gages. — There are two forms of gages in general use: (1) a staff or board gage, vertical if possible, fastened rigidly to the banks or some permanent object, as a tree or bridge abutment; (2) a chain gage attached to some permanent part of the bridge. *A timber gage* is generally a plank painted white, graduated to feet and tenths, preferably by means of galvanized iron staples. It is not always possible to arrange a staff gage in one section to take all variations in water stage, but this, of course, does not especially matter if the two or more sections are not too far apart up and downstream and are referred to the same datum. A gage of any type should always be arranged so that minus readings will never have to be made, and it is usually advisable for this reason to assume zero at several feet below any probable low water.

Chain Gages. — The advantages of the chain gage are that it is completely above water and not within reach of ice, logs or freshet conditions. The gage as developed by the Geological Survey, as shown in Plate II, consists essentially of a standard weight arranged for perhaps 0.2 or 0.3 ft. adjustment at the end and a heavy sash chain, the whole being arranged inside of a long box, with a downspout at one end when the gage is not in use. It is essential that the length of chain be known and kept constant, and the reading of the gage checked frequently by means of the bench marks as noted previously. The bench marks to which the gage is referred must, of course, fill the general requisites for such reference points. Frequently a copper plug is set in rock above high water and the gage referred to that. The datum of the gage is also referred to some known datum such as mean sea, whenever that can be done without too great expense.

Cable Stations. — In case a bridge is not available, and the span is not too great, a cable can be stretched across the stream where conditions are good, and measurements made from a box or car operated on this cable. It is essential that the cable be

PLATE II.


13'-0"
— Lengths —
A = 13'-0"
B = 12'-7 3/8"
C = 13'-1 3/4"
Chain Index Point
— SCHEDULE OF MATERIAL FOR ONE BOX —
LUMBER:—
A, A, A = 1 piece 6 1/4" x 16'
B, B = 1 " 3 3/4" x 16'
C, C = 1 " 5 1/4" x 16'
All 7/8" planed white pine
HARDWARE:—
Chain (No 1 std. heavy sash)
Chain weight
Ring and 2 S hooks
Wire staples for graduations
(12-2", 12-1", 100-3/4")
8-7/8" flat-head screws
Nails.
1" Compo. metal numbers, 0 to 10 incl.
1-3" sash pulley
3-4" heavy strap hinges
1-8" hinge hasp
21-3/16" wire tire bolts, 1 1/4" long
1 std. padlock, key and staple
Note:— All material, except lumber and nails, supplied from Washington office
CROSS SECTION
1/2 SIZE
DEPARTMENT OF THE INTERIOR
U.S. GEOLOGICAL SURVEY:—HYDROGRAPHIC BRANCH
STANDARD CHAIN GAGE BOX
AS USED IN NEW ENGLAND
DISTRICT HYDROGRAPHER
BOSTON, MASS.
MAY 1, 1905.
PIECES FOR SPOUT
Make 1
Make 1
Make 2
END OF BOX
Make 1
Paint outside of box and inside of cover thoroughly, usually red.
Take especial care to paint hinges and hasp. Attach end spout
at place where gage is to be used. In shipping, place pieces
of spout either inside box or nailed to cover, and fasten
weight and chain etc. securely in box, locking with padlock

high enough above the stream so that the bottom of the car shall be several feet above any possible flood height, and the cable is usually held in place either by fastening to trees or to posts which are set in the ground and properly braced. A wire, tagged in a similar way to a surveyor's chain, is stretched across above the cable a few feet, to show distances from a reference point on shore. Excellent results can usually be obtained at a cable station. Its disadvantages are the cost of installation and the somewhat increased difficulty of making measurements. If the gage is at any considerable distance upstream or downstream from the point where the cable is, it will be necessary to maintain an auxiliary gage near the cable so as to properly compute soundings for high-water conditions.

Boat Stations. — Where no bridge is available and the stream is very wide, measurements have to be made from a boat, and in this case it becomes necessary usually to provide stay-lines from both bow and stern and to arrange a tagged line to locate soundings, as is the case of cable stations. The usual method is to have the boat pointed upstream and the meter cable sustained from the bow by means of an outrigger and pulley. The objection to the use of a boat is that some motion is unavoidable, and consequently inaccuracies are apt to occur in both soundings and velocity measurements. In many cases, however, it is the only practicable way of making measurements.

Gage Reader. — The desirable object, as regards observations for a record of river height, is usually to make such a number of readings as will enable the mean gage height to be known for any one day. This result is usually obtained by having two readings taken, twelve hours apart, — in the morning and at night. It is, of course, necessary to supplement this routine by occasional extra gage readings during periods of flood in order to catch the high points. Where the conditions of flow are interfered with frequently, as by pondage from dams, it is sometimes difficult to procure a set of gage readings in any one day that will correctly represent the mean for twenty-four hours. In some cases a single reading taken early in the morning will give the best results, while in other cases it becomes desirable to make the readings at particular times of the day and not necessarily at equal time intervals. Where the flow is very much complicated artificially, it is sometimes necessary to install an automatic gage in order to know at all closely what the mean gage height is for any short period of time. The chief requisites of a good gage reader are faithfulness and reliability, as the reading of

the gage is not a matter requiring special training. Usually postmasters of small towns, or railroad station agents, make very good readers, as their duties compel them to be always on hand.

Current Meter Discharge Measurements. — Before starting in to make a current meter measurement it is always desirable to check the gage reading in some manner; for instance, the length of chain (if it be a chain gage) is measured, and adjustment made to correct this if necessary. A measurement is made to the water surface from a bench mark on the bridge, the height of which is known according to the gage datum, to still further check the reading of the chain gage, and this same method will check the reading of a staff gage. The routine of a discharge measurement is then commenced, and, for ordinary and low stages, soundings are made by the current meter itself by means of a cloth tape. The number of points at which velocity measurements are taken will, of course, depend upon the width of the stream. A general rule is as follows: If the width of the river is less than 25 ft., take observations every 2½ ft. or less; if width is more than 25 ft. and less than 200 ft., take observations every 5 ft. or less; when more than 200 ft. and less than 500 ft. wide, take observations every 10 ft. or less; if more than 500 ft. wide, take observations every 25 ft. or less. Soundings are made whenever it is possible, in order to keep a record of changes in the condition of the river bed. But at high-water stages, it is frequently impossible to secure good soundings, and under those conditions it becomes necessary to compute the soundings, or rather to compute the areas corresponding to the particular gage height, from a standard cross section of the river at that point, which is made up from the soundings of low and ordinary stages with some additional data obtained by levels at the banks.

Current Meters. (See Fig. 2 and 3.) — There have been many forms of current meters used by the Survey in one way or another, but it has been found that the Price electrical current meter gives, in general, best results for river work. This meter is made in two sizes, the smaller size for use at ordinary and low-water stages, and the larger size, which is built stronger, for use when velocity is high and the number of revolutions of the small meter would become too great for counting. The meter is suspended by a double conductor covered with No. 14 or 16 flexible copper wire, which is heavily insulated. When this cable is new and in good condition it will safely carry lead to the amount of 35 or 40 lb., but it is generally desirable to employ an additional rope when more than about 30 lb. of lead is on the

Fig. 1. Cable Gaging Station on St. Croix River, at Sprague's Falls, Me.

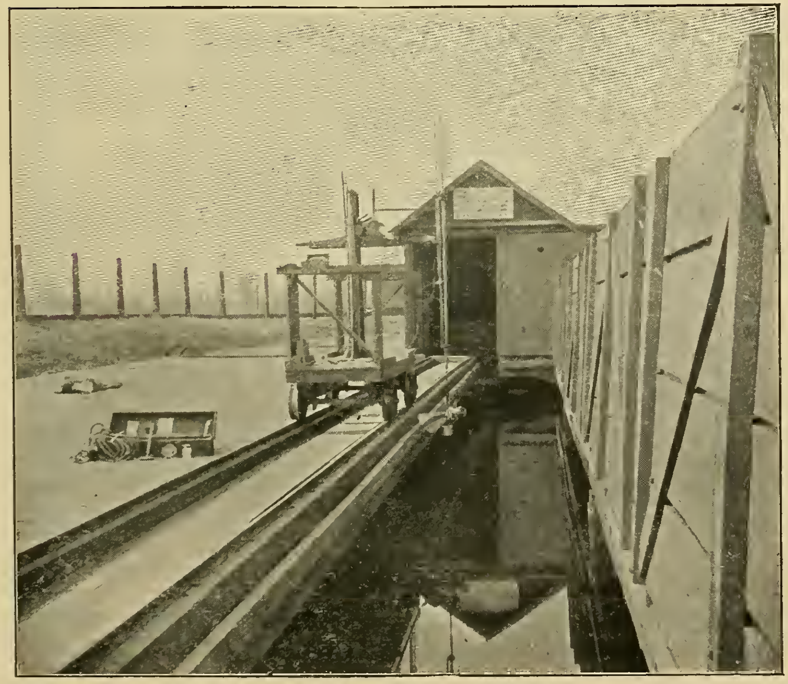


Fig. 4. Current Meter Rating Station at Denver, Colo.

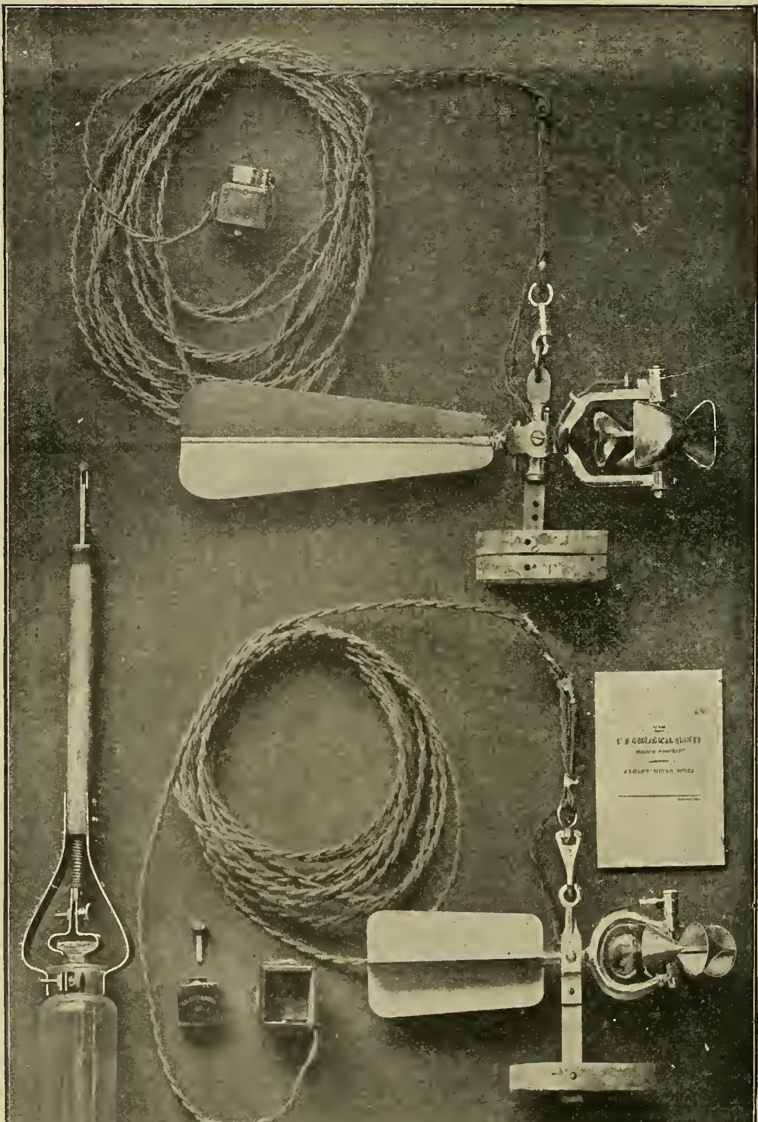


FIG. 2. PRICE ELECTRICAL CURRENT METER.

meter. The general arrangement is shown by the cross section of the small meter. The cable is attached to the meter by a spring snap hooked into the smaller end of the trunnion, and the heavy copper wires are connected to the meter binding posts *h* and *d* by smaller and more flexible wires, arranged loosely to permit the meter swinging freely in a vertical plane. The lead weights are attached to the lower end of the trunnion by means of a detachable weight stem. When any considerable amount of lead is used with the small meter, it becomes necessary to use an

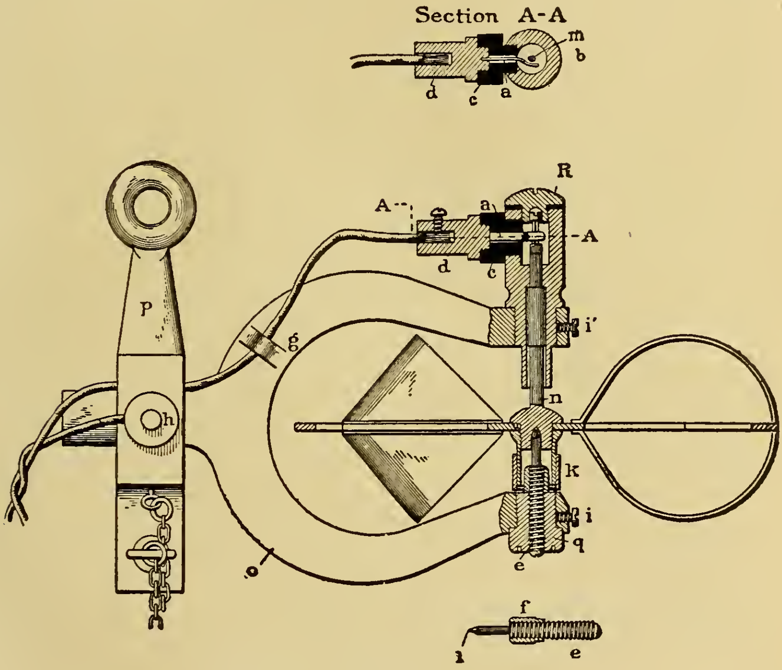


FIG. 3.

additional vane to hold the meter and keep it pointing up stream. It is to be noted that the Price meter turns upon a vertical axis, and the greater part of the bearing comes on the lower point, which is adjusted by the thread at the end, and held in place by means of the circular nut *q*. The vertical axis to which the cups of the current meter wheel are attached, terminates at the lower end in an inverted cone which bears upon this point. In order that no shock come upon the bearing point when the meter is being carried from place to place, a sleeve nut is arranged to be screwed down against the frame, and consequently lift up the cups

and shaft and bring the bearing off of the point. It is, of course, very necessary in using the meter to insure that the sleeve nut is sufficiently far up so as not to interfere with the free motion of the meter. The device for indicating revolutions of the meter wheel is shown in the cross section of AA. *c* is a circular piece of hard rubber for insulation, connecting with one of the small wires from the cable, entering the middle binding post at *d*, which extends through the hard rubber insulation and terminates in a slender platinum spring *a*. The top end of the shaft of the meter at *m* is slightly bent so as to give it an eccentric motion as the cups revolve; thus making and breaking the electric circuit as each revolution is made. The battery used is small and compact in form, and is the result of a great deal of experiment and trial. It consists of a hard rubber cell with a cylindrical-shaped carbon, which is connected with a platinum point extending through the bottom of the cell. The other pole is of zinc extending through a rubber stopper which closes the cell. The exciting medium is bisulphate of mercury, of which about one-half teaspoonful is used, moistened with water. This battery cell fits in a case of which several different forms are employed, and so arranged that connection is made from two poles with a form of buzzer, so that as the circuit is made and broken a buzzing sound is made. This form of apparatus gives more or less trouble, owing to the various parts of the buzzer getting out of adjustment, but so far it is the most satisfactory form devised. It has, at least, the advantage of being exceedingly compact in form, and is placed at the end of the cable near the observer.

Rating of Current Meters. (See Fig. 4.) — The manner of rating the meters in use by the Survey is shown by the accompanying photograph of the current meter rating station at Denver, Colo., and consists essentially of a car with which by means of suitable fixtures the current meter can be readily held and moved along in still water at various known rates. The process, of course, is an inverse one from that used in measuring the velocity of flowing water. An automatic rating device for current meters is now being designed whereby the two factors, velocity and motion of the meter, and the number of revolutions of the meter will be recorded on a strip of paper side by side, and thereby greater accuracy secured in this feature of current meter work.

Improvements of the Current Meter. — Improvements of the current meter are being made from time to time. Two important features now being considered are, (1) the use of ball bear-

ings, which should make the low velocity readings much more reliable, and (2) the use of some device whereby, when desirable, only every fifth (or tenth) revolution of the meter will be recorded. This will enable the small meter to be used for high velocities, and result in a greater field of usefulness for it.

Measurements of Velocity. — It is usually desirable to measure the velocity in each vertical where a sounding is taken, unless the change is small between points and the velocity of flow regular. The essential at any vertical section is to obtain the mean velocity in that section. The general methods in use may be classified into three: (1) single point, (2) multiple point, and (3) integration.

Single-Point Method: Three single-point methods have been used. In the one most common, called the 0.6-depth method, the meter is held at the depth of the thread of the mean velocity; in another, called surface observation, the meter is held about one foot below the surface; in the third the meter is held at mid-depth. In all of these methods it is necessary to apply a coefficient to reduce the observed velocities to mean velocities. The first method has the advantage, however, that the coefficient is very nearly unity, provided conditions are good at the measurement station. The depth of mean velocity is not constant, and will perhaps vary from 0.55 to 0.70 of the depth under such conditions, depending on the depth, the ratio of width to depth and roughness of the bed. The method of surface observations has to be used at very high stages, as under such conditions it is impossible without special appliances to hold the meter at any considerable depth. The meter is, however, held about one foot below the surface and a coefficient varying from 0.85 to 0.90 or more applied to velocities. The third method of holding the meter at mid-depth is now little used.

Integration Method: In this method the meter is kept in motion either from the surface to the bottom and back again in a vertical line, or diagonally from the surface to the bottom and back again, while it is at the same time moving across the channel. This latter method cannot, however, be successfully employed under the usual river conditions. In the vertical integration method satisfactory results can be obtained if the meter is moving slowly and at a uniform rate of speed, and it is occasionally very useful under such conditions as measurements of velocities underneath the ice, or measurements of velocities in flumes and canals where the flow is not a true gravity flow.

Multiple-Point Methods: (1) Vertical velocity curves; (2) top and bottom; (3) top, mid-depth and bottom; (4) other combinations of velocities at various depths.

This first method of taking sufficient observations to enable a vertical curve to be drawn should be used when there is time for it, as it is the most accurate in its results. It is rarely possible, however, to make a complete gaging by this method owing to the liability of fluctuation in gage height while the measurement is being made; for, in order to obtain suitable vertical curves, numerous observations usually have to be made in each vertical. It is very largely used, however, as a check upon all the other methods.

(2) The top-and-bottom method gives very variable results. In the first place the two terms "top" and "bottom" seem indefinite and liable to variations, depending on the meter used and amount of lead, etc. Then, too, the coefficient to use under these conditions is not well known, nor is it at all constant.

(3) In the top, mid-depth and bottom method the mean velocity V is taken as $\frac{1}{4}$ (T + B + 2 M) where T, B and M represent the observed velocities at top, bottom and mid-depth respectively. This is, of course, based upon the assumption that the vertical curve is composed of two straight lines meeting at mid-depth, and is only approximately correct.

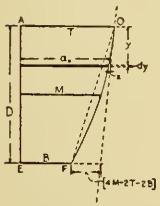


FIG. 5.

(4) Other combinations. (*a*) Upon the assumption that the curve of vertical velocities is of parabolic form with horizontal axis, an expression for the mean velocity in terms of surface, bottom and mid-depth velocities is $V_m = \frac{1}{6}(T + B + 4M)$, using notation as previously.

$$\text{For area AOEF} = \int_0^D (a_x - x)\, dy =$$

$$\int_0^D \left[T - \frac{y}{D}(T - B - 4M + 2T + 2B) - \frac{y^2}{D^2}(4M - 2T - 2B) \right] dy$$

$$= TD - \frac{D}{2}(3T - B - 4M + 2B) - \frac{D}{3}(4M - 2T - 2B)$$

$$= TD - \frac{5}{6}TD + \frac{1}{6}BD + \frac{2}{3}MD = \frac{D}{6}(T + B + 4M)$$

$$\therefore V_m = \frac{\text{Area}}{D} = \frac{1}{6}(T + B + 4M). \qquad (1)$$

This method seems to apply very closely to the usual types of vertical velocity curves, and the assumption of parabolic form of curve is well substantiated by curves obtained under various conditions of flow. In applying this method a slight approximation is necessitated in field work occasioned by impracticability of getting observations of velocity at the surface or on the bottom. Usually a depth of 0.3 ft. to 0.5 ft. is the nearest point attainable to the surface, while owing to the weight standard, etc., the lowest observation must be made some 0.5 ft. above bottom.

(*b*) In studying some vertical velocity curves under ice conditions, with the idea of obtaining results with fewer observations, it was found by the writer that the mean velocity was quite closely a mean of the velocities at 0.2 and 0.8 total depth. This seemed to apply well under various conditions, and moreover it was found that the same relation held for open-water conditions. It is, perhaps, of interest to compare this method with the preceding, assuming the velocity curve as parabolic as before. Referring to the previous figure, the velocity at any depth is:

$$(a_x - x) = T - \frac{y}{D}(3T + B - 4M) - \frac{y^2}{D^2}(4M - 2T - 2B)$$

For 0.2 depth, $y = 0.2\,D$.

$$\begin{aligned} \text{and } V_{0.2} &= T - 0.6\,T - 0.2\,B + 0.8\,M - 0.16\,M + 0.08\,T + 0.08\,B \\ &= 0.48\,T - 0.12\,B + 0.64\,M. \end{aligned}$$

For 0.8 depth, $y = 0.8\,D$

$$\begin{aligned} \text{and } V_{0.8} &= T - 2.4\,T - 0.8\,B + 3.2\,M - 2.56\,M + 1.28\,T + 1.28\,B \\ &= -0.12\,T + 0.48\,B + 0.64\,M. \end{aligned}$$

$$\text{Hence, by this method, } V_m = \frac{V_{0.2} + V_{0.8}}{2} = \frac{0.36\,T + 0.36\,B + 1.28\,M}{2}$$

$$= 0.18\,T + 0.18\,B + 0.64\,M.$$

$$\text{or } V_m = \frac{1}{6}(1.08\,T + 1.08\,B + 3.84\,M). \qquad (2)$$

Thus the difference between mean velocities by (1) and (2) is:

$$\frac{1}{6}(-0.08T - 0.08B + 0.16M).$$

It is evident that when B = T = M this difference is zero, and that for the usual types of vertical velocity curves it will be small. The advantages of this method over the other are: (1) fewer observations in the vertical; (2) the meter can be

placed exactly at 0.2 and 0.8 depth; (3) simplicity of computation.

Examples of Vertical Velocity Curves and Comparison of Methods.

Following is a diagram showing three sets of curves for different conditions of the Winooski River near Richmond, Vt.

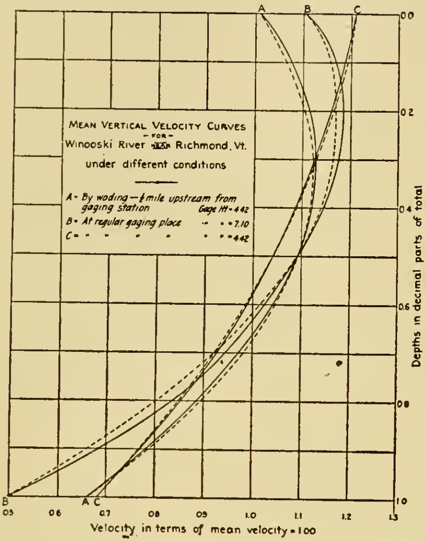


Fig. 6.

Curve "A" is based on twelve curves obtained November 11, 1903, by wading at a point about one-half mile upstream from the bridge, from which the other series were taken. The average depth was 1.44 ft. and mean velocity 2.39 ft. p. s. Bottom of river of gravel 1 in. to 5 in. in diameter, a typical section for measurements by wading.

Curve "B" is based on seven curves obtained May 4, 1904, at regular station; gage height = 7.10, which corresponds to a medium high stage of river. At this point the bed is sandy. Average depth = 11.5 ft. Mean velocity = 2.64.

Curve "C" is based on three curves obtained November 11, 1903, at regular station; gage height = 4.42, corresponding to low water.

In each case a parabola, shown by dotted lines, is drawn for comparison, passing through top, bottom and mid-depth points of curve and with axis horizontal. It is seen that with curve "B" the greatest variation from parabolic form occurs, and moreover the parabola is wholly on one side of the curve as observed. In the other two cases the curves approximated quite closely the parabola; also the variation is about evenly distributed on each side.

The following table summarizes results:

Series.	Coefficients for Reducing to Mean Velocity.						Thread of Mean Velocity.
	6/10 Depth.	Top and Bottom.	Top.	$\frac{T+2M+B}{4}$	$\frac{T+4M+B}{6}$	$\frac{0.2+0.8}{2}$	
A	0.958	1.199	0.992	1.037	0.993	0.998	0.589
B	0.976	1.247	0.903	1.057	1.006	0.990	0.633
C	1.012	1.056	0 826	1.006	1.007	0.995	0.580

The following is a summary of several sets of vertical velocity curves made during 1904, under a considerable range in conditions:

Place.	Date, 1904.	Average Depth.	Average Velocity.	No. of Curves.	Coefficients for Reducing to Mean Velocity.						Depth of Thread of Mean Velocity.
					6/10 Depth.	Top and Bottom.	Top.	$\frac{T+2M+B}{4}$	$\frac{T+4M+B}{6}$	$\frac{0.2+0.8}{2}$	
Aroostook River, Me. . . Bed gravel — ordinary.	May 10,	12.0	5.5	3	0.996	1.103	0.925	1.039	1.016	0.997	0.63
Mattawamkeag River, Me. . Bed rough and rocky.	April 15,	8.2	4.9	3	0.972	1.187	0.958	1.049	1.009	1.000	0.67
Piscataquis River, Me. . . Bed *very rough.*	April 22,	3.6	3.56	4	0.941	1.179	0.971	1.035	1.000	1.007	0.69

To further compare some of the above methods, the following data have been arranged from the results of vertical velocity curves as published in Water Supply Paper No. 76.

TABLE . VELOCITY CURVES ON CATSKILL MOUNTAIN STREAMS, 1902.

(See W. S. & I. Paper No. 76, Table XV, p. 45.)

STREAM.	Esopus.			Rondout.	Wallkill.	Catskill.	Fishkill.	Ten-mile.	Housatonic.	Average of all the Streams.
No. of Velocity Curves .	12	8	20	13	9	14	13	5	4	78
Bed of Stream . . .	Small gravel	Bowlders	Average	Bowlders	Silt	Small gravel and rock ledge	Large gravel	Sand	Gravel	Various
Mean Depth . . .	6.5	4.7	5.8	6	7.3	3.2	3.7	5	4.9	5.07
DEPTH BELOW SURFACE IN PARTS OF TOTAL.	VELOCITIES IN TERMS OF MEAN VELOCITY, represented as 1.00.									
0.20	1.199	1.160	1.181	1.191	1.169	1.233	1.136	1.142	1 141	1.171
0.80	0.803	0.838	0.819	0.796	0.832	0.763	0.861	0.866	0.845	0.826
$\frac{0.2 + 0.8}{2}$	1.001	0.999	1.000	0.993	1.000	0.998	0.998	1.004	0.993	0.998
$\frac{T + 4M + B}{6}$	1.004	1.005	1.004	1.012	0.997	1.005	1 005	0.999	1.012	1.006
0.60	0.989	1.017	1.002	1.018	0.996	0.972	1.014	0.981	1.012	1.003

In making computations for the above, the figures for 0.2 and 0.8 depth have been obtained by averaging between those given for 0.05 and 0.15, and for 0.75 and 0.85 respectively. Probably results are slightly small owing to the nature of the curves at these places. For computation by the formula $\frac{T + 4M + B}{6}$ T has been taken as at 0.05 depth and B as at 0.95 depth, which is a fair assumption.

The general indications from such examples as have been considered, show that the parabolic formula in approximate shape, *viz.*, $\frac{0.2 + 0.8}{2}$ is fully as satisfactory in its results as is the theoretically correct one, $\frac{T + 4M + B}{6}$.

Low Velocity Limitations. — A very important feature of current meter work, and one which is not always appreciated, is that of the unreliability of the meter when the velocity of flow is small. Numerous experiments have been made to define the accuracy of various forms of meters under low velocity conditions and have been published in Water Supply Papers of the Survey. The results of these experiments show that the small Price meter measurements of velocities less than about 0.4 ft. per sec. are not reliable, the chief reason being that the friction of the meter on its bearings grows relatively greater as the velocity decreases. It is very frequently necessary to make low-water measurements for this reason at some portion of the river where better velocity conditions can be obtained, and usually this is done by wading, the meter being usually held by the cable in the hand of the observer, and at a short distance from him, so that the flow will be as unobstructed as possible.

In general the intention is to make at the permanent stations of the Survey vertical velocity curve measurements at as many points in the cross section as conditions will permit, and at varying stages of the river, so as to be able to intelligently apply a coefficient to such measurements as have to be made by the two single point methods. Thus, in the case of arriving at a suitable coefficient to use for meter measurements made at high water by the method of surface observations, it is frequently possible to obtain vertical curves at some lower gage heights, and, by studying the coefficients thus obtained, to predict very closely what coefficient should be made for the higher stages.

Computations of Discharge Measurements. (See Fig. 7.) — As the velocity is not constant in all parts of the cross section

the quantity flowing through a given unit of area differs with its location, and hence it is necessary to divide the cross-sectional area into a number of parts and compute the discharge through each part separately. The size of the component parts is determined by conditions in the field as noted previously. There are two methods of computation in general use by the Survey: (1) the single-strip method, and (2) the double-strip method.

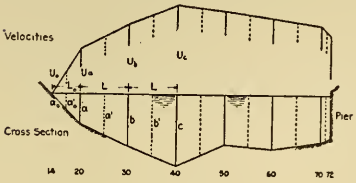


FIG. 7.

In the *single-strip* method the area is divided as shown by dotted lines a', b', c', etc. Soundings and velocities have been observed at sections a, b, c, etc., all a constant distance L apart. The area between a' and b' may be considered as the product of the unit length L by a mean depth d_m. It is assumed that the bed of the stream is a *straight line* between soundings.

$$a' = \frac{a+b}{2};\quad b' = \frac{b+c}{2}$$

$$\frac{b+b'}{2} = \frac{b+\frac{b+c}{2}}{2} = \frac{3b+c}{4};\quad \frac{b+a'}{2} = \frac{b+\frac{b+a}{2}}{2} = \frac{3b+a}{4}$$

$$d_m = \frac{\frac{b+b'}{2}+\frac{b+a'}{2}}{2} = \frac{\frac{3b+c}{4}+\frac{3b+a}{4}}{2} = \frac{a+6b+c}{8}.$$

Any given area $A = Ld_m$.
If V_b = observed mean velocity at b,
discharge through strip $a'b' = Q_b = V_bA_b$.

If the end strip is under consideration, or whenever, owing to piers and obstructions, or for other reasons, it is deemed best to make soundings and velocity observations at unequal intervals, it is necessary to compute each half area, as $a_o'a$ and aa', by itself.

In the practical application of the formula for mean depth a shorter method for computations is used.

$$d_m = \frac{a+6b+c}{8} \quad \text{may be written}$$

$$d_m = \frac{(a-b)+8b+(c-b)}{8}$$

$$= b + \frac{(a-b)+(c-b)}{8};$$

or, in other words, to the depth at the middle of section considered, add $\frac{1}{8}$ of the difference as regards the next section each way, of course strictly regarding algebraic signs.

Thus, if $a = 4.8$
$b = 6.2$
$c = 7.0$

By formula,

$$d_m = \frac{4.8 + 37.2 + 7.0}{8} = 6.12,$$

By modification,

$$d_m = 6.2 - \frac{1.4}{8} + \frac{0.8}{8} = 6.2 - \frac{0.6}{8} = 6.12.$$

The mean depth can thus be figured mentally and very quickly.

The *double-strip* method of computation is shorter in that a double strip is considered at one time. Consider the double strips from *a* to *c* in the previous figure: *a*, *b* and *c* are soundings.

V_a, V_b, V_c are observed mean velocities at *a*, *b* and *c*. The assumption is made in this method that the stream bed is a series of parabolic arcs, which is probably quite closely true. The mean depth d_m for the double strips *a* to *c* is then,

$d_m = \frac{a + 4b + c}{6}$ in a similar way as shown on page 18, for mean velocity. The further assumption is made in this method that the *horizontal* velocity curve is parabolic with axis parallel to the direction of the current. Thus, for velocity,

$$V_m = \frac{V_a + 4\,V_b + V_c}{6}.$$

Hence the discharge for double strip is

$$Q_{ac} = d'_m V_m 2L = \frac{a + 4b + c}{6} + 2L\frac{(V_a + 4V_b + V_c)}{6}.$$

If a single strip is under consideration, as at a pier or near the bank, then

$$V_m = \frac{V_o + V_a}{2};\ d = \frac{a_o + a}{2}$$

in which case V_o and a_o may be zero.

Following is a double page of the current meter notebook now in use, with computations by both of these methods These notes correspond to the section shown in Fig. 12, and furnish an instance where considerable difference occurs in results by the two ways. In general, however, these two methods give substantially the same results.

Gaging made ...., 190 , by .... Meter No. ....

Gage height: beginning ....ft., ending ....ft., mean ....ft. River rising, falling, sta.

6—563

Dist. from initial point.	Depth.	Observations.				Velocity computations.				Computations of	
		Depth of observat.	Time in seconds.	Revolutions.		Total number revolutions.	Revolutions per second.	Velocity per second.	Mean velocity per second.	Width.	Mean depth.
"SINGLE STRIP" METHOD.											
14	0.0	—						0.		3.	
20	2.4	1.4	↑	15	16		0.31	0.75		8.	
30	4.2	2.5		24	25		0.49	1.17		10.	4.18
40	5.8	3.5	50 secs.	32	31		0.63	1.49		10.	5.38
50	4.0	2.4		30	29		0.59	1.40		10.	4.28
60	4.4	2.6		27	27		0.54	1.28		10.	4.28
70	3.8	2.3	↓	24	25		0.49	1.17		6.	
72	3 5			abt.	same			1.0		1.	
										58.	
"DOUBLE STRIP" METHOD.											
14									0.38	6.	1.20
20											
30									1.15	20.	4.17
40											
50									1.40	20.	4.37
60											
70									1.22	10.	4.10
72									1.08	2.	3.65
										58.	

Rating Curves. (See Plate III.) — From a list of gage heights and corresponding discharges, as computed from a number of current meter measurements, the station rating curve is constructed. The usual method is to take discharge in cubic feet per second as abscissas and gage heights as ordinates. The curve thus obtained is usually parabolic in form, at least for lower gage heights, although for higher stages of the river in most cases it develops into a straight line or nearly that. It is most convenient usually to have the curve running at about an angle of 45 degrees and the vertical and horizontal scales are chosen accordingly. If the conditions are satisfactory at the gaging station, the various points usually lie quite closely to a

On..............................River, at..................................., State of..............................

Total area....................sq. ft. Mean velocity.................. Discharge.................. second-feet.

Area of Sec.	Discharge of Section.	REMARKS
Area.		On condition of channel, wind, equipment, gage, boat, cable, methods, accuracy. Use cross-section pages in back of book for sketches.
		Length of gage chain checked and found to be—
1.8	0.0	Right bank.
19.6	14.7	
41.8	48.9	
53.8	80.2	
42.8	59.9	
42.8	54.8	
23.4	27.5	
3.6	3.6	Pier.
229.6	289.6	

7.2	2.7	
83.4	95.9	
87.4	122.4	
41.0	50.0	
7.3	7.9	*Computed by*..
226.3	278.9	*Checked by*..

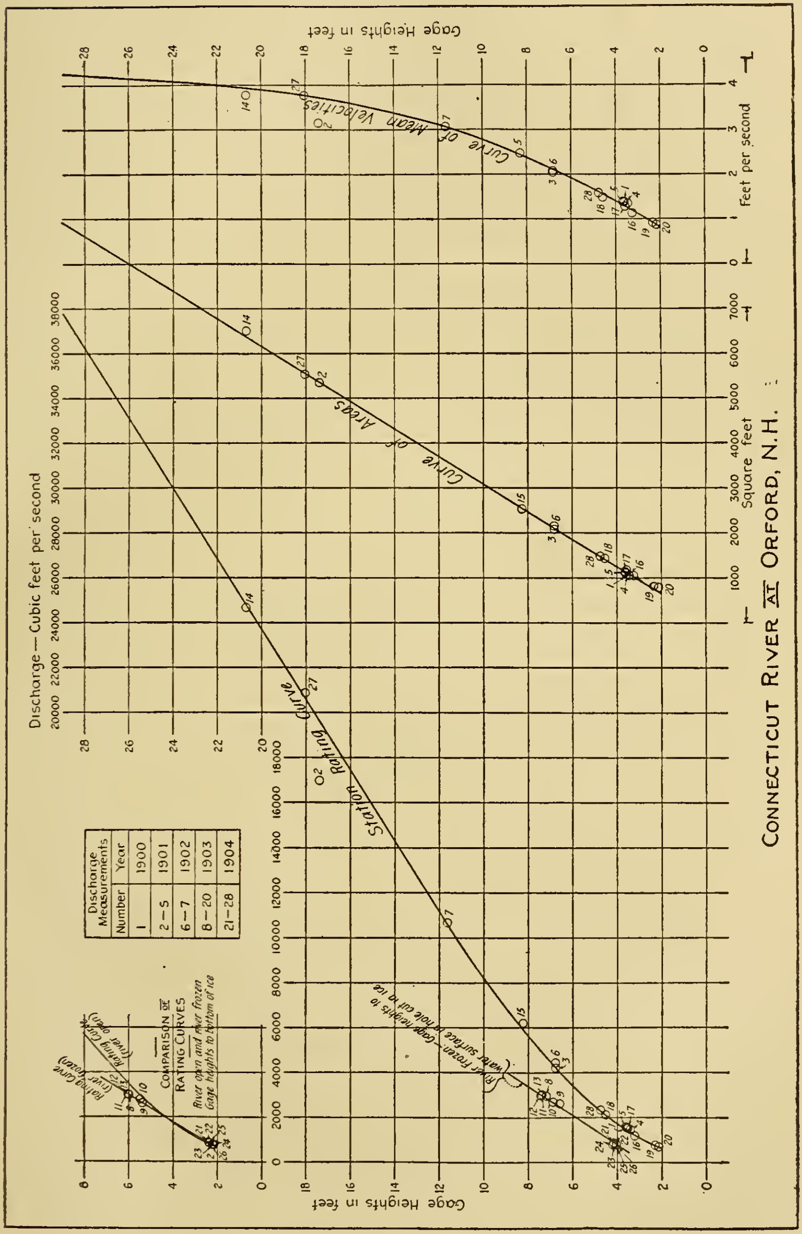
mean curve drawn through them; but if conditions are poor, the bed of the stream rough, or other difficulties are met with, there may be considerable variation in the position of some of the points as regards the curve. In such cases, however, the points should be distributed on both sides of the curve, so that the resulting error is perhaps about only half of what it would be in case of a single point measurement. A general rule now in operation with regard to a single measurement is that, when the percentage of error is five times the average percentage of error in all the other measurements, it shall be rejected.

The assumption is made in applying this rating curve that the discharge at any given gage height will always be the same.

This, of course, is not always strictly true. If we consider the general formula representing stream flow $Q = AC\sqrt{RS}$, it is evident that if any one of the terms making up this formula change between two successive positions of the river at the same gage height that a different discharge will result for that gage height. The quantity A (area) of the cross section must not change; that meaning that there must be no scour in the vicinity of the gage and, on the other hand, there must be no filling in of material. The coefficient C depends upon several factors, chiefly the conditions of the banks and bed of the stream, and these, of course, may vary from time to time. Thus, in some streams during the summer season there may be a growth of grass or vegetation that will markedly affect this coefficient. The hydraulic radius, of course, refers to the cross section of the stream for some little distance above and below the measurement point, and it is evident that this quantity, which consists of the area divided by the wetted perimeter, must remain constant, and this means, of course, practically permanent conditions if the rating curve is to apply. The last factor in the formula, that of slope, S, must also be considered. It is very possible to have a greater or less slope for the same gage height, depending upon the stage of the river. For instance, if the river is rising very rapidly the slope may be considerably steeper than it would be for the same gage height with a steady condition of flow, and so a greater discharge result. The matter of river flow is, of course, a very complicated one, and the construction of a rating curve from current meter measurements must be made with a great deal of care and study of the conditions of the various measurements, so as to give a proper weight to each one.

In order to make estimates of flow for gage heights beyond the reach of actual meter measurements, it is sometimes necessary to extend the rating curve. The discharge at any gage height may be considered as the product of the area corresponding to that gage height by a mean velocity. It is possible to estimate very closely what the area will be for any gage height. This can be done graphically by constructing a curve of areas as compared to gage heights, and, of course, by taking levels on the banks of the river this curve can be extended to any extent desirable. The other factor, that of velocity, can be treated in a similar way, but in that case, of course, the curve will have to be extended by using simply the curve of velocities resulting from such measurements as have been made. Thereby an estimate of each of these two quantities for any gage height

PLATE III.



CONNECTICUT RIVER AT ORFORD, N.H.

beyond the limit of measurements on the rating curve can be made, and their product will give the corresponding discharge. This eliminates one of the uncertainties and calls for an estimate simply as regards velocity, and is the common method employed where extensions of rating curves are necessary. Of course a fair idea of the discharge can be obtained by extending the discharge rating curve. In its upper portions it is usually a straight line, as previously noted, and this is readily extended. In its lower portion it is usually of parabolic form, and it is frequently advisable to make the extension on logarithmic cross-section paper, and thus transform the curve to that of an approximate straight line to facilitate its extension.

Rating Tables. — For convenience in use the rating table is constructed from the rating curve. The first draught of this table is taken by scaling from the curve. It is then smoothed out by adjusting first, and, if necessary, second differences, keeping in mind the fact that the difference between discharges for successive tenths should either be constant or increasing, but never decreasing. The precision with which this table is constructed will depend upon the conditions at the station. In the case of small streams it is frequently necessary to carry gage heights to every 0.05 ft. or less, and the corresponding discharges to the nearest second-feet, while the large streams are usually carried to the nearest 0.1 ft. of gage height, and perhaps 5 or 10 sec.-ft. discharge will do. This rating table is then used in connection with the list of gauge heights for any given year to construct a table of estimated daily mean flow at the station, and from this the various factors of mean monthly flow, run-off in inches on the drainage basin and run-off per square mile of drainage basin are computed.

Rivers with Unstable Condition of Bed or Banks. — The problem of determining daily discharge of streams with changeable beds is a difficult one and requires very frequent discharge measurements if the results are to be other than mere approximations. For such conditions discharge measurements often have to be made every two or three days, and discharges for intervening days obtained by interpolation modified by gage heights for those days. Where the changes are slower, or occur perhaps during floods, rating curves can be made to apply for the period of time between changes, and perhaps satisfactory results obtained by two or three meter measurements a month. These conditions do not in general occur here in the northeast. In some few cases, however, there have been marked changes in

the condition of the bed of the stream at the station due to spring floods, and consequently a revision of the rating curve has been necessary.

Estimates of Flow when River is Frozen. — The foregoing methods apply only when a river is under conditions of open water. One of the weak points in the current meter method of estimating discharges, and in fact a weak point in any other method, is the matter of estimates during the frozen season. It is evident in the general formula for flow, that all of the various quantities are going to be changed by ice conditions. Thus the area will be cut down and a portion of the space in the cross section filled with ice, which may be floating more or less freely; or, on the other hand, if the stream be a narrow one, it may be arched between the banks and largely self-supporting. Under conditions of winter flow, the river is practically in a closed conduit, and at times the flow may be under a considerable head. In addition to friction on the bed and perhaps the banks of the stream we have friction on the bottom of the ice covering, and it is evident that this latter will vary considerably with the condition of the ice.

The gage height is read by cutting a hole in the ice. There is some question perhaps as to what point the gage reading should be taken, to the bottom of the ice, or to the surface of the water as it rises in the hole. The method in use by the Survey at present under winter conditions is to have the gage reader make an observation perhaps once or twice a week, noting the gage height to the water surface, and the top of the ice, and measuring the thickness of ice, this latter being done by means of a graduated stick with angle iron at one end. Current meter measurements of flow have to be made by cutting holes through the ice at proper intervals, and, as is evident, the current meter measurements under such conditions are very much more difficult and expensive to make than in the open season. The thread of mean velocity under such conditions does not remain anywhere nearly at a constant depth, and thus far most such measurements have been made either by taking vertical velocity curves or by the method of vertical integration. The first method is preferable, for, although it takes considerably more time, it is more accurate and provides all the means for a study of the flow under such conditions.

The rating curve as constructed for river frozen is quite different from that in the open season. The work of making systematic measurements of flow during the winter season has

only been going on a short time, and in fact only at one New England station are there sufficient data yet obtained to enable rating curves for open water and frozen conditions to be compared. By reference to the rating curve for Connecticut River at Orford, N. H. (see Plate III), a comparison may be seen of the rating curves for open and closed channel within a limited range of gage heights. In one case where gage heights are taken to the bottom of the ice, the rating curve for ice conditions crosses the ordinary curve, although coinciding fairly closely with it in the lower portions. In general the discharge is less for the higher gage heights than in the case of the ordinary curve. When gage heights are taken to the water surface, and a hole cut in the ice, the resulting curve as shown is entirely separate from the ordinary curve. It would seem as though this last method would prove the most reliable, for it is possible in the case of the smaller streams to have the ice actually held in place by adhering to the banks, so that the position of the bottom of the ice would not be a true index of the amount of water flowing. If a hole is cut in the ice it seems fair to believe that the height at which water will stand in this hole bears some relation to the amount flowing.

The present status of estimates of stream flow during the winter season is very largely an experimental one, but it is hoped to devise methods for continuing the records of flow throughout the whole year at a reasonable cost.

In the foregoing discussion, the writer has intended to outline briefly the work that is being done in New England by the Hydrographic Branch of the United States Geological Survey, and to give a résumé of the current meter method of estimating river flow, as now in use. This paper is largely based upon the publications of the Survey descriptive of these methods, and their accuracy, and a list of the Water Supply Papers which are especially pertinent to this subject, as well as those reports of stream measurements which contain data on New England rivers, follows.

While more detailed descriptions will in many cases be found in the references given, the writer feels that these governmental publications are not generally known about and consulted to the extent that they should be, and so takes this opportunity, at the risk perhaps, of some repetition of the data already published, to call the attention of engineers to the broad scope of this work, and, in addition, to present a basis for further detailed discussion.

United States Geological Survey Publications Relating to General Hydrographic Investigations.

Nineteenth Annual Report, Part IV, 1897, pp. 17–31. "Methods of Measurement."

Twentieth Annual Report, Part IV, 1898, pp. 20–22. "Methods of Measurement."

Twenty-first Annual Report, Part IV, 1899, pp. 28–41. "Methods of Investigation; also Difficulties in Measuring Flow over Dams."

Water Supply Paper 35, 1899, pp. 11–25. "General Discussion of Methods."

Water Supply Paper 47, 1900, pp. 10–15. "Methods of Using Stream Gagings for the Computation of Water Power."

Twenty-second Annual Report, Part IV, 1900, pp. 49–50. "Methods of Investigation."

Water Supply Paper 56, 1901. "Methods of Stream Measurement."

Water Supply Paper 64, 1902. "Accuracy of Stream Measurements."

Water Supply Paper 76, 1903. "Observation on Flow of Rivers in Vicinity of New York City."

Water Supply Paper 80, 1903. "Relation of Rainfall to Run-off."

Water Supply Paper 94, 1904. "Hydrographic Manual."

Water Supply Paper 95, 1904. "Accuracy of Stream Measurements." (2d ed.)

United States Geological Survey Publications Relating to Hydrography in New England.

Nineteenth Annual Report, Part IV, 1897, pp. 34–117.
Twentieth Annual Report, Part IV, 1898, pp. 45–47; 64–78.
Water Supply Paper 27, 1898, pp. 9–16.
Twenty-first Annual Report, Part IV, 1899, pp. 50–63.
Water Supply Paper 35, 1899, pp. 25–44.
Twenty-second Annual Report, Part IV, 1900, pp. 56–81.
Water Supply Paper 47, 1900, pp. 29–36.
Water Supply Paper 65, 1901, pp. 13–42.
Water Supply Paper 69, 1902, "Water Powers of Maine."
Water Supply Paper 82, 1902, pp. 11–58; 77–79.
Water Supply Paper 97, 1903, pp. 14–103; 338–355.
Water Supply Paper, 124, 1904 (in print).

GRADE-CROSSING PROBLEMS IN ST. LOUIS.

By Carl Gayler, Member of the Engineers' Club of St. Louis.

[Read before the Club March 1, 1905.]

There are several reasons why I bring this subject before the club. In the first place its great importance; then the fact that, to my knowledge, it has never been treated in its different bearings, and last, not least, the circumstance that in studying the question I have arrived at solutions of two of the most important of the problems, which I consider as very satisfactory, namely, of the Tower Grove Avenue crossing of the Missouri Pacific and Frisco tracks, and of the grade crossings of the Wabash and Colorado roads west of Kingshighway.

To do away with a crossing on grade of one of the streets inside of our city limits by railroad tracks is a problem the difficulty of which has been increasing from year to year. Greater valuation of land, delays and expenses arising from claims for damages, real or imaginary, to owners of adjoining property, etc., have made the building of viaducts more and more expensive and slow. The time has come — in fact it arrived quite a number of years ago — when very little can be accomplished in the grade-crossing question unless both parties, the city and the railroads, act together. Laws passed by the city, throwing the whole burden on the railroads, will fail; on the other hand it is not fair if, as heretofore, the railroads partake of the advantages without sharing in the expenses.

That the benefits derived by both parties are great and permanent is so self-evident that they do not need to be enlarged upon. Saving of life and limb, greater speed of trains, no interruption of street traffic, etc., are things everybody can appreciate at their true value. There is one benefit, however, to the railroads alone, to which special attention is called. It is the opportunity of improving the grade of their tracks which the abolition of a grade crossing, in most cases, gives to the railroads. To this important point I shall have to refer again and again.

Before going on with the subject, a short review of the history of railroading in this city, as far as it has a bearing on the question, will not be out of place.

The first legislative enactment on the part of the state of Missouri on a steam railroad entering this city bears the remarkably early date of 1837, and is entitled: "An act to incorporate the St. Louis & Bellevue Mineral Railroad." The road was probably never built, and, in 1851, its charter was merged into that of the St. Louis & Iron Mountain Railroad. The Pacific Railroad was chartered in 1849, which charter was amended in 1851. About the same time a charter was granted to the North Missouri Railroad Company.

There is a charming simplicity and comprehensiveness in the wording of these laws. The St. Louis & Bellevue act reads: The company is empowered "to locate the route for a railroad for a single or double track, the same to be not more than 100 ft. wide, commencing in or near the city of St. Louis, etc., and to build said road along or across any state or county road, and the streets and wharves of any city, town or village, whether corporate or otherwise, and over any public stream or highway, the consent of the county court having been first obtained." The last clause is important, and it seems to have been carefully omitted in the charters of the three other roads. The amended charter of the North Missouri Railroad, for instance, reads: "The corporation is empowered to survey, mark out, locate, construct and continue its road from the city of St. Charles *to any point in the city of St. Louis*." The wording of the charter granted to the Iron Mountain & St. Louis Railroad is similar, and the amended charter of the Pacific Railroad empowers the company to "survey, mark out, locate and construct a railroad *from the Mississippi River or any other point in the city of St. Louis, to any point on the western line of this state*." A former city councilor has stated that it is this sweeping clause of the charter granted by the legislature of the state to the Pacific Railroad which has defeated all efforts on the part of the city to have the oldest and one of the most objectionable of the grade crossings, the Poplar Street tracks, removed.

Thus, in the early fifties, were the present Missouri Pacific Railroad, the St. Louis, Iron Mountain & Southern Railroad and the Wabash Railroad launched out on their future career of greatness; one also sees plainly in these enactments the glad hand given by the people and their representatives to these harbingers of a new era, with not a shadow of fear of future complications.

However, a few years after these charters had been granted

by the legislature, one finds further and more carefully prepared legislation on the part of the city. In 1854 the Pacific Railroad is granted permission to extend its line from its Fourteenth Street depot to Seventh Street, the Iron Mountain & St. Louis Railroad to construct its line from the southern city limits, at that time near Keokuk Street, to Mulberry Street, and the North Missouri Railroad to construct its line from North Market Street northwardly to the city limits on Grand Avenue. In these city ordinances is found already some provision for the safety and convenience of street traffic in the following section: "Provided said railroad shall not be so constructed as to prevent the public from using said street, . . . and that said company shall keep that part of the street used by them in good repair, and they shall in all cases provide for the convenient crossing of their track or tracks for wagons, drays and other vehicles by planking or paving up to the level of its rails, or within one inch of said level; and any changes of grade to be made only with the consent of the city engineer." Then follows a provision empowering the city to enforce the above. The rate of speed is limited to four miles per hour and the engine bell is to be tolled through the city limits. It was not till twenty-five years later, March, 1879, that ordinances were passed providing for safeguards at railroad crossings.

It may seem strange that, in view of their unlimited charters, the railroad companies submitted to these regulations on the part of the city, but this can be well understood when we read that at this time financial support was given by the city to each of the three railroad companies.

As all the street crossings were on grade it follows that the gradients of these early roads were strictly governed by the established grades of the streets which they crossed. Since the Iron Mountain and the North Missouri roads were laid down on streets near to and parallel to the river, on what may be called contour lines of the west shore of the same, tolerably good grades were obtained. It was different, however, with the Pacific Railroad, which is an east and west line. The profile (Fig. 1) of a part of this road, from Eleventh Street to Ewing Avenue, shows this.

From now on, all rights and franchises within the city limits were granted by the city of St. Louis, and they follow, chronologically, about as follows:

Poplar Street tracks, connecting the Pacific Railroad with the Iron Mountain & St. Louis Railroad in 1872; Kirk-

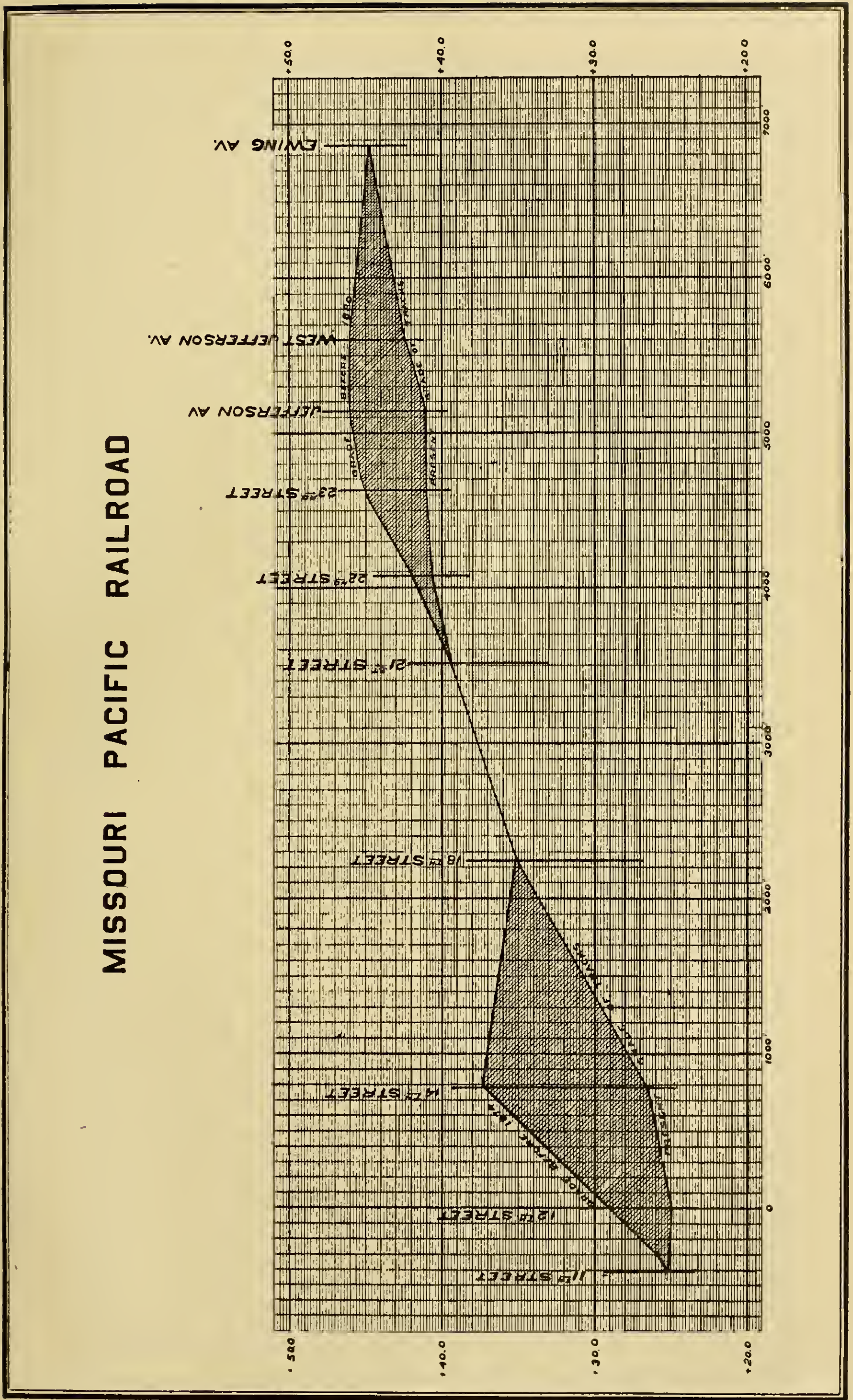

MISSOURI PACIFIC RAILROAD
EWING AV.
WEST JEFFERSON AV.
JEFFERSON AV
23RD STREET
22ND STREET
21ST STREET
18TH STREET
14TH STREET
12TH STREET
11TH STREET

FIG. 1.

wood Branch of the Missouri Pacific, 1872; Union Depot and tunnel, 1874; Wabash Line from western part of the city to Union Depot, 1875; Frisco right-of-way from the city limits to Tayon Avenue, 1881; Oak Hill Road, 1886; Colorado Railroad, 1887; Merchants Bridge Elevated and bridge, 1887; Terminal belt line, 1889; St. Louis, Keokuk & North Western, 1887–1889.

The opening of the Eads Bridge on the Fourth of July, 1874, and the passing of the ordinance within one week after the opening of the bridge, authorizing the building of a union depot on Twelfth Street, mark a new era. Provision is made in the ordinance for the building of viaducts on Twelfth and Fourteenth streets, and, what is probably of equal importance, for a uniform grade of the depot tracks from Eighteenth Street to the mouth of the tunnel and a corresponding depression of the tracks of the Missouri Pacific Railroad. The benefit of a better gradient resulting from this law is shown on the profile (Fig. 1) of the latter road. I may overestimate the value of such grade reductions, but I confess that in all railroad work nothing appeals to me so much as the endeavors to make grades easier which have been going on for years all over the country, particularly on eastern roads. The advantage derived may be small in each case, but it is everlasting, and, in the case under consideration to-day, over thirty tracks are benefited by the change.

The same profile shows another improvement in the railroad grades, made possible through the building of the East Jefferson bridge in 1880, and the story of the same is rather a curious piece of history.

To obtain easy grades for the proposed viaduct a raising of Chouteau Avenue of 5 ft. and a lowering of the railroad tracks were necessary; and, considering the chance to reduce their grades, it should have been expected that the railroad companies would have welcomed the opportunity. Instead of that, a fierce fight ensued between the city authorities and the managers of the Missouri Pacific Railroad, and the protection of 50 policemen was required to enable the men to put in the foundation piers of the bridge on the railroad's right-of-way. The lowering of the tracks was not carried out as originally planned, but a compromise was made. (Profile.)

The conception and successful completion of this scheme, at least as far as the city's interests were concerned, is one of the things which have made the memory of Colonel Flad dear to us.

It is worth while to examine the city ordinance, under which the tracks on Jefferson Avenue were lowered. Said ordinance *authorizes* the lowering of the tracks; it does not *compel* the railroad companies. It reads: "Whereas, the construction by the city of the East Jefferson Avenue viaduct over tracks of the Missouri Pacific and Wabash, St. Louis & Pacific Railroads, makes it necessary for these companies to depress their tracks at this point, and

"Whereas, *the lowering of the tracks of these roads at High Street and Jefferson Avenue will lessen the grade between these crossings*, therefore . . . ." In other words, the act is based on the equity of the case; it recognizes, and the railroad companies are likewise expected to recognize, the mutual benefits arising both to them and to the city of St. Louis, by the building of the viaduct and the lowering of the tracks.

In the case of new railroad lines built in the city under franchise granted by the city authorities from about 1880 to the present day, it must be acknowledged that here the interests of both the city and the railroad companies have been well guarded. For instance, in building the Oak Hill line Kingshighway south of Arsenal Street was crossed overhead, and highway bridges were built at several important crossings. On the new western Wabash line there are overhead crossings on the Lindell Drive in Forest Park and on Vandeventer Avenue, and bridges on Kingshighway and Euclid Avenue. The Merchants Bridge elevated along the levee and from the levee to Eighth Street is a standard structure, well worthy of being studied, and, above all, of being repeated. Speaking of repeating it, brings to mind the surface tracks along Poplar Street, referred to above. On the Terminal Belt Line are three well-designed viaducts with permanent floors.

It is worth noting that in the franchise of the Terminal Belt Line is found, for the first time, the important clause that the viaducts shall be maintained by the railroad company. There is, then, an evolution in municipal legislation on grade crossings as well as in organic life.

With the exception of the Kingshighway Bridge over the Wabash Railroad, which was paid for jointly by the railroad company and by the commissioners of Forest Park, all these bridges were built at the expense of the railroad companies. It may also be stated, as a matter of record, that the north ends of the Fourteenth Street and Jefferson Avenue bridges and

of the West Jefferson Avenue footbridge were extended by the railroad companies at their expense.

Of altogether different character has been the treatment of cases where existing grade crossings had to be dealt with. The policy of the city in regard to the same has been simplicity itself; appropriation for new viaducts followed appropriation, until, in 1892, half a dozen viaducts and a footbridge across the railroad tracks in the Mill Creek valley were completed. Beside occasionally suggesting some change in the location of piers or the length and grade of the viaducts, no part was taken by the railroad companies in this work. The total amount of money expended by the city on the building of viaducts across railroad tracks, from the adoption of the charter, in 1876, to date, including their maintenance is, approximately, $1 800 000.

From 1892 to the present day the only appropriations made by the city for new viaducts were for the work done last year on Kingshighway, Union Boulevard and Ewing Avenue, and these would not have been made but for the World's Fair. This suspension of work on new viaducts since 1892 is the more striking since at about that time rapid transit, the strongest ally of the advocate of abolishing grade crossings, was being universally substituted for the mule car (the first cable road here was built in 1889), and serious accidents at grade crossings kept public attention awake on the question.

As stated, the city has spent the sum of $1 800 000 on the work of abolishing grade crossings by means of viaducts. The sole contribution on the part of the railroad companies consists in the money paid into the city treasury by the St. Louis & San Francisco Railroad, namely, $15 000 on completion of each additional viaduct built over its tracks since it entered the city on its own line in 1881. Two such payments have been made up to date,—the sum of $1 800 000 paid by the city against the sum of $30 000 paid by the railroad companies toward abolishing existing grade crossings since the time that the city was reorganized under its charter.

This short survey of by-gone days being completed, the problems will now be treated upon which, sooner or later, action will have to be taken. There are a number of grade crossings, and with the building up of the city, their number will keep on increasing, where, as far as one can judge, viaducts will be built by the city, as heretofore, with little or no assistance from the railroad companies; as instances, the Chouteau

Avenue crossing of the Missouri Pacific Railroad and the Sarah Street crossing of the Wabash Railroad.

There are other crossings where the coöperation of the railroads, in the shape of changes in their lines and gradients, is a necessity. To this class belongs, for instance, the Ewing Avenue crossing, where a viaduct with serviceable grades could not be built unless the railroad tracks are lowered.

There is still another class of grade-crossing problems,— those where the greater burden of the work falls on the railroad companies, which burden they are expected to take up in consideration of lasting, direct benefits; the city, in these instances, probably assuming the expenses arising from slight alterations in street grades, reconstruction of streets, sewers, etc., and such a share of the cost as will be deemed just. To this class belong the Tower Grove Avenue crossing and the Wabash and the Rock Island crossings west of Kingshighway. The general plans, profiles, etc., show the solution of these two problems.

Fig. 2 shows the proposed improvement at the Tower Grove Avenue crossing. The tracks are those of the Missouri Pacific and the Frisco Railroads; the intersected streets are Tower Grove Avenue and Old Manchester Road, the latter with the car tracks of the Tower Grove Avenue line and the Southampton line. It will not be necessary to dwell on the dangers and delays of this crossing.

As the neighborhood is well built up and the railroad tracks are not located in a depression, as in the Mill Creek valley, approaches of viaducts along Tower Grove Avenue and Old Manchester Road would be of great length. Preliminary plans and estimates have shown that their great cost and the delays in acquiring the necessary land and in adjusting claims for damages practically exclude the possibility of building viaducts over these crossings. A glance at the profile of the railroad tracks shows that an elevating of the railroad tracks over and across the street is, even more than the building of viaducts, out of the question. The profile of the railroad tracks, however, shows also that any depression of the same is an improvement of their grade, and following out this idea I have prepared the plans of a Tower Grove Avenue and Old Manchester Road subway, which I beg leave to submit for judgment.

Earlier in this paper the improvement of railroad gradients, which, in many instances, is obtained by separating the street and railroad traffic, was spoken of, and of this the Tower

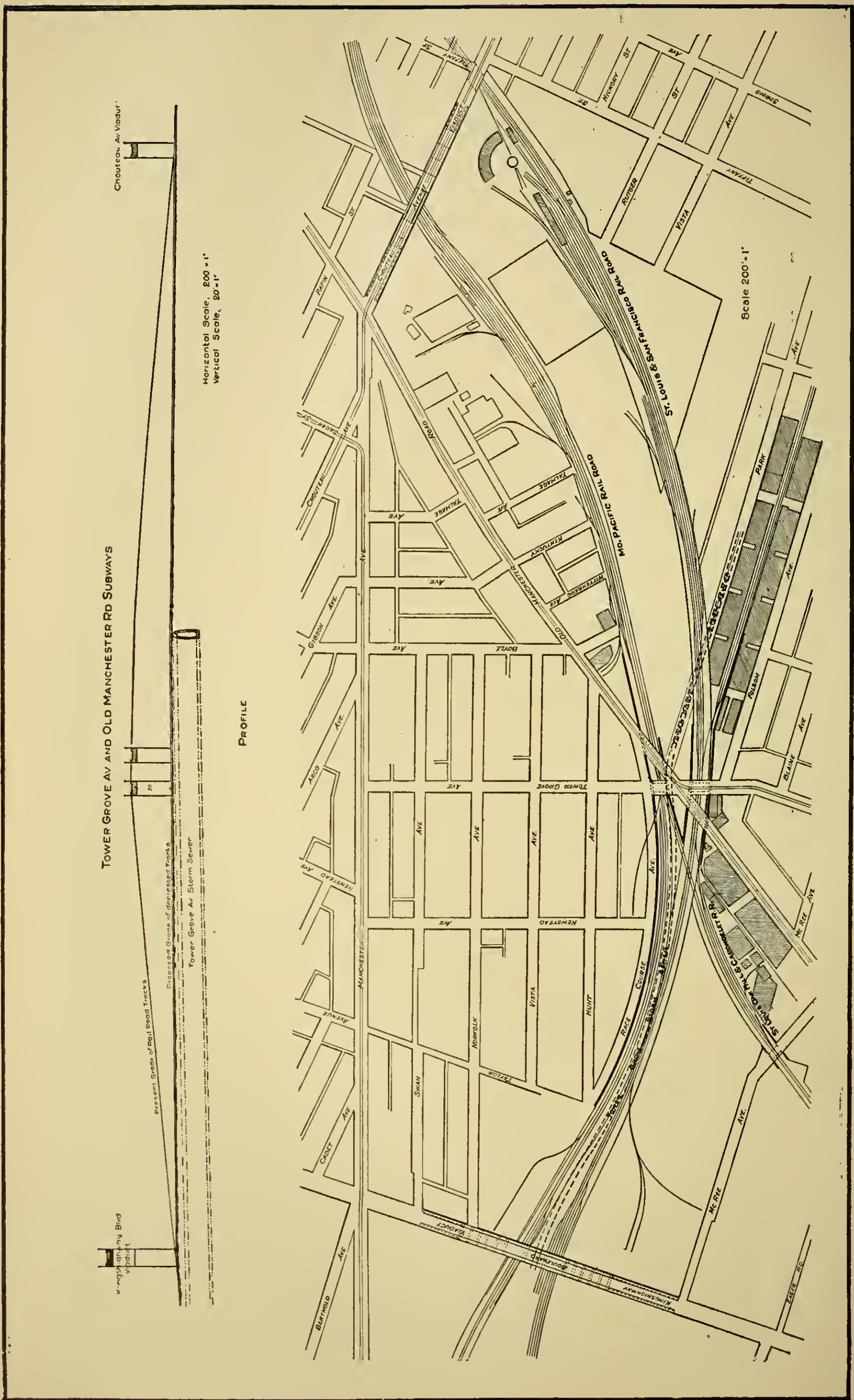
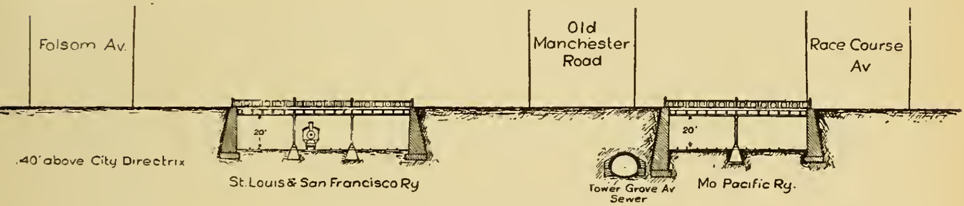

TOWER GROVE AV AND OLD MANCHESTER RD SUBWAYS
PROFILE
Horizontal Scale, 200 = 1"
Vertical Scale, 20 = 1"
Chouteau Av Viaduct
Kingshighway Blvd Viaduct
Present Grade of Rail Road Tracks
Proposed Grade of depressed Tracks
Tower Grove Av Storm Sewer
Scale 200' = 1"
MO. PACIFIC RAIL ROAD
ST. LOUIS & SAN FRANCISCO RAIL ROAD
CHOUTEAU AVENUE VIADUCT
OLD MANCHESTER ROAD
MANCHESTER AVE
TOWER GROVE AVE
NEWSTEAD AVE
KINGSHIGHWAY BOULEVARD VIADUCT
RACE COURSE AVE
McREE AVE
PARK AVE
VISTA AVE
TIFFANY ST
HICKORY ST
RUTGER
TALMAGE AVE
KENTUCKY AVE
BOYLE AVE
GIBSON AVE
NORFOLK
SWAN
HUNT AVE
TAYLOR AVE
CADET AVE
BERTHOLD AVE
FOLSOM
BLAINE AVE

FIG. 2.

Grove Avenue subway problem is a most excellent illustration. The Frisco Railroad crosses Chouteau Avenue at an elevation of 50 ft. (in a round figure) above City Directrix, and Kingshighway at elevation 49 ft.; the elevation of the Missouri Pacific Railroad at both these crossings is 55 ft., and both roads cross Tower Grove Avenue and Old Manchester Road at elevation 76 ft. A straight grade line from Chouteau Avenue to Kingshighway will, therefore, intersect the two street crossings at an elevation over 20 ft. below their present grade. The tracks of two of our great trunk lines climb a 1 per cent. grade to meet trouble, great trouble and delays at an important street crossing, only to descend on the other side with as steep a grade, when it is feasible to avoid the grade crossing by a uniform, nearly level grade from a point near Chouteau Avenue to some point near Kingshighway, and by building short bridges across



General Elevation of Tower Grove Av. & Old Manchester R.R. Subways

the tracks along Tower Grove Avenue and Old Manchester Road.

It is seen in the profile that the Tower Grove Avenue storm sewer is built deep enough not to interfere with the proposed lowering of the tracks. The Oak Hill Railroad, now the main line of the St. Louis, Iron Mountain & Southern Railroad, which branches out from the other tracks near Tower Grove Avenue towards the southwest, when lowered at the same time, makes an overhead crossing of Kingshighway over these same tracks feasible without an increase of the present gradient of this line. This is an advantage of the utmost importance, as it does away with the last remaining grade crossing of the boulevard.

Fig. 2 is meant to present the general idea of the proposed solution of the problem; but it is understood that deviations from the same are to be expected, and that, for instance, a slight raising of the street grades and, in consequence, a lesser

amount of depression of the railroad tracks is not incompatible with it.

I do not wish to be misunderstood: in claiming that a subway is the only correct solution of this grade crossing, the difficulties to be met with are duly appreciated. The expense will be considerable, and a number of interesting problems have to be solved in re-adjusting the lines and grades of the switches which now lead from the railroad tracks to the important manufacturing establishments in the neighborhood. The street and railway traffic must also be kept uninterrupted during the progress of the work. But there is not the least reason to doubt that these difficulties will be overcome by the engineers of the railroads and of the city.

This scheme was submitted to the mayor of the city in the fall of 1903, the problem next to be treated in the fall of 1904. Both plans have, however, been more fully elaborated since.

The tracks of the Wabash Railroad Company (see Fig. 3) from the mouth of the tunnel under Kingshighway Boulevard to the city limits now cross Lindell Drive in Forest Park overhead on a beautiful viaduct, abruptly drop to the grade of Lindell Avenue at the north line of Forest Park and then cross Union Boulevard and every other street and avenue, as far as the city limits, on grade. The Rock Island Railroad (formerly the St. Louis, Kansas City & Colorado Railroad) is a surface road from its junction with the Wabash Railroad near Union Boulevard to the city limits near the Washington University grounds. It was again an examination of the profiles of the railroads which suggested the plan of avoiding these grade crossings which is here outlined. This plan consists in a continuation of the present easy ascending grade of the Wabash Railroad from Kingshighway to the Lindell Drive viaduct, on an elevated structure or solid earth embankment with viaducts over the intersecting streets, as far as the city limits, near the Olive Street Road. The Rock Island Railroad is likewise to be raised, in a similar manner, from Union Boulevard to the city limits.

The practical difficulties are in this case smaller than those met with in the proposed Tower Grove Avenue subway. The number and importance of side tracks and switches is not as great; in fact, the proposed track elevation is not more than what has been carried out in numerous other cities.

If this plan is carried out, the two principal driveways which enter Forest Park from the north, Union Boulevard

and De Baliviere Avenue, — each 100 ft. wide, — Lindell Avenue, — the fine driveway along the north line of Forest Park, — and a

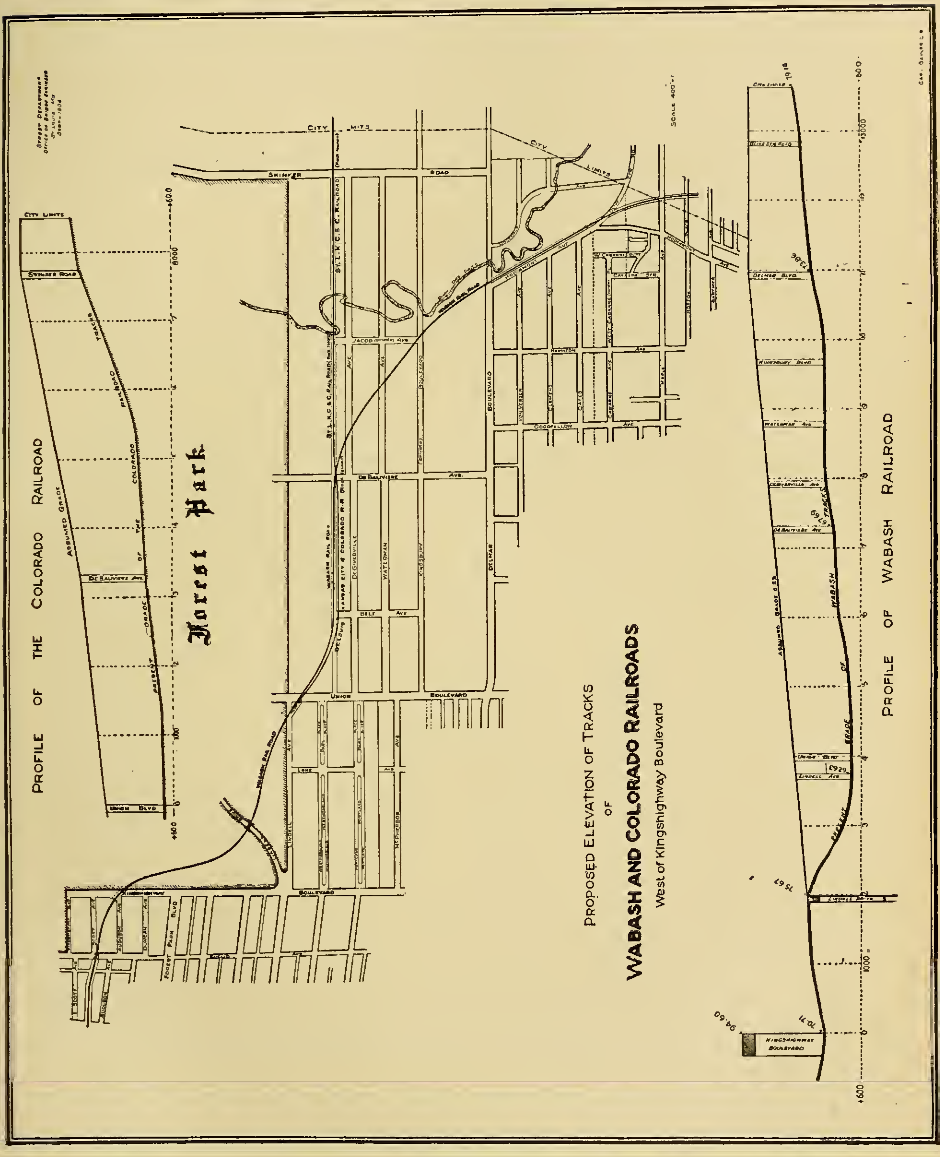


FIG. 3.

number of other intersected streets, including Delmar Avenue, with its important rapid transit traffic, will all be cleared of railroad tracks, and their present light and uniform grades

unaltered. Furthermore, the electric car line along Skinker Road will be safely and promptly conveying, beside the general public, its precious loads of teachers and pupils to the great new seat of learning on Washington University hill. On the other hand, the two railroads, the Wabash and Rock Island, will be freed from delays, freed from expenses for guards and safety appliances, and rid of the bitter chance of maiming and killing people. Surely such a plan should be worthy of earnest consideration.

The grades of the two roads, shown in Fig. 3, are to be considered as suggestions limited by the condition that a height of 14 or 15 ft. should be the minimum clearance over the street crossings. Uniform gradients were thus obtained of 0.30 ft. in 100 ft. for the Wabash Elevated, and 0.40 ft. in 100 ft. for the Rock Island Elevated road.

If it were not for the great importance of the subject, the consideration of other methods of solving the problem could be dismissed in a few words. But the aim is not to show that the proposed track elevation is "a consummation devoutly to be wished," but that it is the only available method of abolishing all grade crossings west of Kingshighway and north of Forest Park. Other plans shall therefore be fully considered.

To throw the whole burden on the city, to expect the city to build viaducts over these numerous crossings, is so obviously unfair, and their cost, direct and indirect, in view of the value of the land and their necessarily great length and width, and the delays in adjusting claims so great, that no adminstration or board of public improvements would be justified in adopting this policy.

To lower the railroad tracks into open cuts or tunnels without altering street grades would necessitate a steep grade of the Wabash Railroad from Kingshighway Boulevard to Lindell Avenue, the taking down of Lindell Drive viaduct in Forest Park and the substitution in its place of a viaduct over the Wabash tracks with long approaches at the entrance and in the most beautiful part of the park. This would mean nothing less than a complete remodeling and rebuilding of the northeast corner of Forest Park from the Lindell Avenue entrance to River des Pères. Add to this the troubles to be expected by the railroads from high waters in River des Pères, and the serious problem of draining the railroad cuts or tunnels, and one will agree that the scheme is not practical.

A combination of the two schemes has been suggested; *i. e.*, it has been proposed to lower the railroad tracks into open cuts, to raise the grades of the intersected streets and to carry them over the railroad cuts on suitable viaducts, but it can be shown that such a plan would partake of the objections made against both of the two other schemes. The Wabash tracks on Lindell Avenue and Union Boulevard cannot be lowered without lowering at the same time the tracks on the Lindell Drive viaduct, because a further increase of the gradient of the Wabash Railroad between this viaduct and Lindell Avenue would be very objectionable, if not out of the question. In other words, Lindell Drive viaduct would have to be sacrificed, and one's imagination can easily picture the nature of the work to be carried out and the damage done to Forest Park, as they have been suggested above. To arrive at a clear understanding of the difficulties to be expected from the raising of the street grades and the building of viaducts, assume at the Union Boulevard crossing a change of grade of 10 ft., a width of the viaduct of 100 ft., an easy grade of the north approach and a somewhat steeper grade on the south approach and in Forest Park; if the railroad companies lower their tracks at their expense, the city's share of the cost will be as follows: compensation to owners of adjoining property for change of grade, cost of condemnation of land along the east and west sides of the Union Boulevard viaduct and along the north line of the Lindell Avenue viaduct, the cost of building the two viaducts and of remodeling part of Forest Park. There is, however, still another side to this question of raising Union Boulevard and Lindell Avenue to be considered, which may not be strictly an engineering problem, but yet I would like to present it, and to do so forcibly, allow me to use an illustration. Transfer, for a moment, the whole situation to New York City; substitute Central Park for Forest Park, 156th Street for Lindell Avenue and Fifth Avenue for Union Boulevard. It is true this is equivalent to magnifying the importance of the case, but why not look forward to a time when the parallel will not look so very much strained? Would New York destroy the easy, uniform grade of an important entrance to Central Park, destroy and rebuild a portion of Central Park, without any gain whatever to the use and beauty of the Park and incur the greater share of the expense of all this work for the sole end of relieving a railroad company from the task of building a few miles of elevated tracks?

Some of these objections apply with equal force to the De Baliviere crossing, and, as far as the numerous other crossings in this district are concerned, the fact has to be noted that none of them is located conveniently in a depression and that each one of them would require long approaches.

The whole grade-crossing problem can be summed up thus: It is, above all, the topography which dictates the proper treatment of each case. Where the railroad tracks are located in a valley, as, for instance, in the Mill Creek valley or on Kingshighway south of Manchester Avenue, viaducts have to be built; where they climb hills on which streets are located, the subway is the correct plan; in districts more or less level, or with gentle slopes, as in the district between Kingshighway and the city limits, and Forest Park and Cabanne, the Chicago plan of track elevation has to be adopted. This is sound engineering, or, what amounts to the same thing, common sense.

In some respects it would have been tempting to speak of the work done by other cities on their grade-crossing problems, and I have, of late, collected considerable material on this subject; but it must be confessed that it might be unpleasant; comparisons would suggest themselves which would be "odious." It may be stated, however, that the Philadelphia subway, built two years ago by the city of Philadelphia at a cost of $6 000 000, half of which sum was paid by the Philadelphia & Reading Railroad, is of the character of the proposed Tower Grove subway, of course on a much larger scale, and that the work done on track elevation in Chicago during the last ten years is a gigantic parallel of the proposed track elevation of the Wabash and Rock Island roads.

Whenever our municipal authorities and the managers of our railroads join hands, and, in a spirit of fairness and of appreciation of the great and lasting benefits to be derived by both parties, start these two works and carry them through, I dare say that our city need not look any longer with a feeling of envy on what other cities, a number of them of smaller size than St. Louis, have, of late years, accomplished in abolishing grade crossings.

I wish to express my thanks to Mr. L. B. Vella, my assistant, for valuable assistance in the working out of these problems.

DISCUSSION.

Mr. Chas. F. Müller. — I agree with Mr. Gayler on the three principal points: First, abolition of all grade crossings as soon as possible; second, earnest and honest coöperation of city and railroad companies to accomplish this result; third, the benefit to all parties will far outweigh the expenses.

With details it is different. Mr. Gayler's two designs are carefully worked out, but I consider them only as specimens showing how other ideas should be worked out. Different engineers will have different opinions and ideas, and, after comparison of designs showing these ideas, it will be possible to find the best solution. Take, for instance, the Wabash Railroad through Forest Park and out to the western limit of the city. In my opinion any plan not throwing the railroad track entirely out of the park ought not to be considered. For one moment imagine the beautiful view from Kingshighway at the main entrance down the gently sloping lawn to the small lakes, and again up the wooded hills; now imagine this view unobstructed by an ugly, high railroad embankment, by the smoke of passing trains, and you have the most beautiful of the many fine views in the park. No railroad would dare to ask permission to put the present obstructions there again if once they were removed, at least as long as we have public-spirited and nature-loving citizens able to fight for the interest of the community. Such an undesirable obstruction must be removed.

Finally, I believe the best and cheapest solution of the problem can only be found from a general plan or plans with approximate estimate of cost. This plan ought to be worked out by city and railroad, embracing all crossings within the city limits, and afterwards competitive general designs ought to be invited. The question will then be in a shape which everybody can understand, and sensible action on it is possible.

It is to be hoped that Mr. Gayler's paper will have the effect of inducing the city to take the initiative soon.

Mr. H. J. Pfeifer. — The elimination of grade crossings is a question that will, sooner or later, have to be faced by the city of St. Louis and the railroads centering therein, and its proper solution will become more urgent and expensive every year that it is delayed. New industries will be located, and new tracks and other appurtenances will be constructed along the railroads; real estate values will increase, and valuable improvements will be made along the streets from year to year.

A change in grade on either railroad or street will involve large damages to abutting property, and the cost of adjusting improvements to the new grade will be heavier every year.

Whether the highway is to go over or under the railroad, whether the grade of the highway or railroad is to be changed, or whether they are both to bear part of the change in grade are problems that must be solved for each particular case. In the two cases that the author mentions, it seems to me that he has hit upon the simplest solutions in that they involve the least interference with existing conditions, improve materially the railroad grades, and interfere very little, if any, with the street grades.

There is one point, however, that it is well for the engineer who has problems of this kind to meet, to keep in mind, and that is that the total change in grade between street and railroad is materially less if the railroad is put overhead than if the highway is put overhead. The clearance required by a man standing on a car is not less than 21 ft. above the top of the rail. In some states the clearance is fixed at 22 ft. by statute. Highway clearance need not exceed 15 ft., and 14 ft. will do if one is in close quarters.

A modern highway bridge floor is as deep if not deeper than a railroad bridge floor; we will assume, however, that they are of equal depth. A minimum difference, therefore, of 6 ft. height in favor of having the railroad cross over the highway is found. It would seem, therefore, in general, where the ground is comparatively level on each side of the railroad or where the railroad runs on a ridge, that the highway should be placed underneath, and that the highway should be placed overhead only where the railroad is already in or can easily be placed in a depression.

Mr. A. H. Zeller. — The subject of this paper is one of keen interest not only to engineers, but also to all the inhabitants and the various interests of the city; and the present time is indeed very opportune for bringing this matter to the attention of the club.

Mr. Gayler has covered the subject very fully, and deserves great credit for the solutions of the two most important grade problems, *i. e.*, the Tower Grove crossing and the Wabash and Rock Island crossing west of Union Avenue.

In the case of the Tower Grove crossing, the reduction of the grades will certainly appeal to the railroads as being a great improvement of their lines. At the same time the city will

escape the great expenditures usually incurred in payment for excessive land damages due to long approaches, in cases where grade of street must be raised. The special switch problems referred to in the paper need cause no serious embarrassment in view of what has been accomplished in the case of the Reading subway in Philadelphia.

The elimination of the street crossing on the Wabash and Rock Island railroads west of Union Avenue by elevation of the tracks seems to be the only feasible solution, owing to objectionable grades and the danger of flooding the roadbed from the high waters of the River des Pères, in the case of a depression of tracks; and it is but natural to suppose that the railroad companies would never consent to a treatment of their roadways which would seriously endanger the operation of their lines. The adjoining property owners would undoubtedly prefer a subway, and no one will dispute that a railroad line is a detriment to a residence locality, and that the property immediately adjoining the right-of-way is not as desirable as if the railroad were a thousand miles away. However, the railroad is there, and the problem is to do the best one can with it, and make it as unobjectionable as possible to the adjoining territory, without at the same time interfering with the regular and continuous operation of the road.

It would seem that the elevation of the road would be best for all concerned, and the question then would be to treat the line in the best manner. The writer would suggest an earth embankment between street crossings with slopes sodded and lined on each side with a row of shade trees. In addition, he would suggest a roadway perhaps 100 ft. wide on each side of the right-of-way with grass plats in the center, with another line of trees next to the curb. This would effectually mask the road from the adjoining property and preserve the park-like effect that it is proposed to give this tract of land.

It is to be hoped that the city fathers and interested railroads will confer and thoroughly look into the problem of grade crossings, and that they will adopt a policy to be pursued from year to year, which will result in the complete elimination of all grade crossings of the main trunk lines within the city limits.

One hears a great deal nowadays about a City Beautiful, and certainly all citizens with the welfare of the city at heart will heartily endorse anything reasonable that will tend in that direction; at the same time it seems to the writer that of greater importance still to the inhabitants of the city is safety to life

and limb, and the abolition of grade crossings is a great step in that direction.

Mr. Gayler. — In the verbal discussion which followed the reading of the paper, the one objection raised against the scheme of elevating the Wabash and Rock Island tracks was the unsightliness of the long earth embankments. This point was more fully referred to in the written discussion of Mr. A. H. Zeller, who suggests a plan of giving the elevated lines a park-like appearance which is well worth considering.

The author thinks that too much stress has been laid on this objection. Railroad tracks in parks and residence districts are more or less objectionable, whether they are elevated or run in open cuts, but, as Mr. Zeller aptly puts it, "the railroad is there, and the problem is to do the best you can with it." The steel viaducts at the crossings will be rather an attractive feature of the scheme, and the embankments can, in comparison, hold their own against open cuts and sunken roads with the smoke of the engines at the height of the first and second story windows of the residences and spreading over the lawns and shrubberies.

The half-dozen tracks of the Illinois Central in Chicago are carried on earth embankments along the length of Jackson Park, the former site of the Columbian Exposition; they pass Washington Park and the Chicago University grounds (the similarity with our case is really striking). Great credit, and deservedly, was given at the time to the city of Chicago and the management of the railroad for this undertaking, and we have yet to hear of adverse criticisms of this work from engineers, or citizens of Chicago, or visitors to that city on the ground of appearance.

The author does not agree with Mr. Mueller's views. It is not wise to strive after the unobtainable, and the location of the Wabash Railroad through Forest Park may as well be accepted as permanent.

The point brought out by Mr. H. J. Pfeifer that there is at least 6 ft. less height required in raising the railroad tracks over the streets, instead of *vice versa*, is well taken.

In conclusion the author confesses to some disappointment in the discussion. He had hoped that the historical part of the paper would receive more attention, as additional data of interest might then have been brought out.

FORCES DUE TO ECCENTRIC WEIGHTS ATTACHED TO ROLLING WHEELS.

By Calvin M. Woodward.

Presented to the Engineers' Club of St. Louis, May 17, 1905.

I.

The Action upon a Horizontal Track of a Heavy Mass attached to Rigid Rolling Wheel.

Let R be the radius of the rolling wheel, and let P be the center of the mass M at a distance nR from the center of the wheel. Denote by ω the constant angular velocity of the wheel. I is the instantaneous axis; $r=PI$ is the instantaneous radius; and the angles ϕ and θ are sufficiently defined as shown in Fig. 1. The path of P is a trochoid, and in the figure the path at P is convex downward. The radius of curvature is easily shown to be

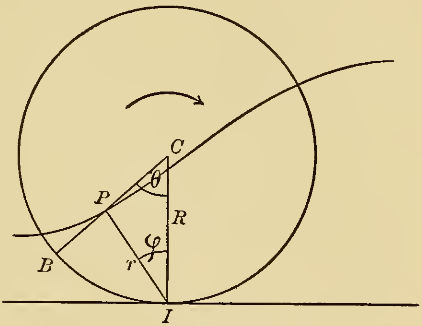


Fig. 1.

$$\rho=\frac{r^2}{R\cos\phi-r}. \qquad (1)$$

The linear velocity of P is $v=\omega r$, and its tangential acceleration is

$$\frac{dv}{dt}=\frac{\omega dr}{dt}$$

But,

$$R^2=R^2+n^2R^2-2nR^2\cos\theta;$$

hence,

$$d\,r=\frac{nR^2\sin\theta\, d\theta}{r};$$

and since

$$\frac{d\theta}{dt}=\omega,$$

we have the tangential acceleration

$$\frac{dv}{dt}=\frac{\omega^2 nR^2\sin\theta}{r}=\omega^2R\sin\phi. \qquad (2)$$

The radial acceleration, or rate of deviation, is

$$\frac{v^2}{\rho} = \frac{\omega^2 r^2(R \cos \theta - r)}{r^2} = \omega^2(R \cos \phi - r). \qquad (3)$$

Now the action of the mass on the track consists of three parts: First, that due to the weight of the mass, which is *W;* second, that due to the linear acceleration, which is

$$\frac{W}{g} \omega^2 R \sin \phi;$$

and the vertical component of its reaction is

$$\frac{W}{g} \omega^2 R \sin^2 \phi. \qquad (4)$$

Third, that due to the centrifugal force which is

$$\frac{Wv^2}{g\rho} = \frac{W}{g} \omega^2(R \cos \phi - r),$$

and its vertical component is

$$\frac{W}{g} \omega^2(R \cos^2 \phi - r \cos \phi). \qquad (5)$$

The sum of these three is the total vertical action:

$$F = W \left[1 + \frac{\omega^2}{g} R \sin^2 \phi + \frac{\omega^2}{g} R \cos^2 \phi - \frac{\omega^2}{g} r \cos \phi\right]$$

$$= W\left(1 + \frac{\omega^2}{g} R - \frac{\omega^2}{g} r \cos \phi\right).$$

But from the figure,

$$R - r \cos \phi = nR \cos \theta;$$

hence,

$$F = W\left(1 + \frac{\omega^2}{g} nR \cos \theta\right), \qquad (6)$$

which is the formula required. This is evidently only the weight plus the downward component of the centrifugal force of *M* revolving about a stationary axis.

Now $\omega R = V$ is the forward velocity of the wheel in ft. per sec.; ω is the angle turned per second, or

$$\omega = \frac{V}{R}:$$

hence, the formula becomes

$$F = W\left(1 + \frac{V^2}{gR} n \cos\theta\right), \qquad (7)$$

in which g may be taken as 32, and n is the fractional part of the radius between the center of the mass and the center of the wheel.

In this formula (7) it is seen that the action of an eccentric weight on the track is greater for a wheel of small radius than for a wheel with a larger radius, n and V remaining the same. That is, with the same eccentric weight placed in the middle of the radius, and the same forward velocity, the "pounding" of the small wheel is twice as great as for a wheel of twice the diameter.

If $n = \frac{1}{2}$ and $V = 88$ ft. per sec. (a mile per min.), and $\theta = 0$, so that $\cos\theta = 1$, and $R = 3$ ft.,

$$F = W(1 + 40\tfrac{1}{3}).$$

This is the track action when the weight is at the lowest point.

If $\theta = 180$ degrees, then $\cos\theta = -1$, and

$$F = W(1 - 40\tfrac{1}{3}).$$

This is the action when the weight is at the highest point; that is to say, the track is relieved of $39\frac{1}{3}$ times W by the lifting action of the mass.

If $n = 1$ the mass is on the circumference of the rolling wheel, and its path is cycloidal. The track action is then a maximum for the wheel; for $V = 88$ ft. and $R = 3$ ft.; the maximum pressure on the track is

$$F_1 = W(1 + 80\tfrac{2}{3}),$$

and the maximum lift when the mass is at the top of the wheel

$$L_1 = W\,(80\tfrac{2}{3} - 1).$$

If W is large it is evident that the action upon the track varies between wide limits during every revolution.

II.

If there be a second weight, W', on the other side of the center and on the same diameter, at a distance $n'R$ from the center, the total action upon the track will be

$$F = W + W' + \frac{\omega^2}{g}\,[WnR\cos\theta + W'n'R\cos(180^\circ + \theta)]$$

$$= W + W' + \frac{\omega^2}{g}\left(WnR - W'n'R\right)\cos\theta. \qquad (1)$$

Now, if

$$\frac{W'}{W}=\frac{nR}{n'R},$$

i. e., if the masses are inversely as their distances from the center, the quantity in the parenthesis is zero, so that $F = W + W'$, and the weight W is perfectly counterbalanced by W'.

If there be two equal counter weights, each W' and each at a distance $n'R$ from the center, and at intervals of 120 degrees, we have

$$F=W + 2 W'+\frac{\omega^2}{g}[WnR \cos\theta+W'n'R (\cos [120^\circ+\theta]+\cos [240^\circ+\theta])]$$

$$=W + 2W' + \frac{\omega^2}{g}\left(WnR - W'n'R\right)\cos\theta; \qquad (2)$$

and if

$$\frac{W'}{W}=\frac{nR}{n'R}$$

as before,

$$F=W + 2W',$$

and W is perfectly balanced by the joint action of the two W's.

III.

It is a simple problem of mechanics to show that the resultant centrifugal force of a mass revolving about an axis is the same as if the whole revolving mass were concentrated at its center of gravity.

This proposition really contains the whole theory of balancing an eccentric weight, viz., it may be balanced by any weight or combination of weights which shall make the center of gravity of them all (including W) at the center of the wheel.

IV.

Action of an Eccentric Mass attached to a Circle which rolls around another Circle, both lying in the same Horizontal Plane.

In the first part of this paper I found the action of a heavy weight attached to a vertical circle rolling on a straight horizontal line.

I found that the result was identical with the result gained by assuming that the axis of the wheel was stationary and the mass was revolving about it with the angular velocity of the

rolling wheel. It at once occurred to me that the general proposition might be illustrated as follows:

In the case of a body having a motion which is the resultant of two motions, the resultant along any line of the forces acting upon the body and causing or controlling its motion is found by adding the components, along the same line, of the forces causing or controlling the two component motions taken separately. In other words: The forces due to component motions may themselves be compounded, as may forces and motions in general. This general proposition of mechanics may be illustrated concretely in the following manner:

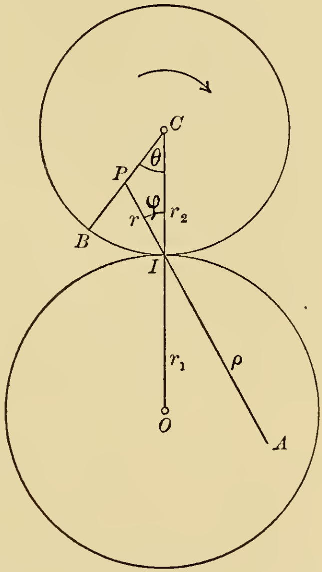


FIG. 2.

Let the mass M be attached to a radius of a circle C, which rolls around the circumference of the circle O, both circles being in the same horizontal plane. Required, the tension along the bar OC while the mass moves along its epitrochoidal path. The circles themselves and the connecting bar OC may be regarded as imponderable, as only the forces acting on the mass at P are to be considered. Let the axis C and the link OC move about the fixed axis O with an angular velocity a, while the circle C rolls with an angular velocity b with reference to OC, and an angular velocity in space of c. Let the radius of the fixed circle be r_1, of the rolling circle r_2.

We have from kinematics that $c = a + b$, and the triple proportion

$$a : b : c = r_2 : r_1 : r_1 + r_2. \qquad (1)$$

Let the center of mass be at P, a distance nr_2 from the center C, and let I be the instantaneous axis of the moving circle. The angles ϕ and θ are sufficiently defined in the figure. The radius of curvature of the path at P lies along PI, and its general length is

$$\rho = \frac{rc}{c - \dfrac{ar_1 \cos\phi}{r}} \qquad (2)$$

in which $r = PI$, the instantaneous radius.

This value of ρ is readily found as follows: The arc described by P in the time dt is $crdt$, in a direction perpendicular to PI. In the same interval a new instantaneous axis is reached at a distance from I of ar_1dt, and its projection upon a line parallel to $crdt$ is $ar_1 \cos \phi \, dt$.

Now these two arcs $crdt$ and $ar_1 \cos \phi \, dt$ are in the ratio of ρ and $\rho - r$, A being the center of curvature of the epitrochoidal curve at P. Hence,

$$\frac{\rho}{\rho - r} = \frac{crdt}{ar_1 \cos \phi \, dt}$$

whence the value of ρ given above.

Now the resultant force acting upon M to control its motion arises from two sources: The velocity of P along its path increases as r increases; hence, so long as θ is less than π, a tangential accelerating force is necessary. Second, P has a radial acceleration due to its moving in a curve; hence a deviating force is necessary.*

The velocity of P in its path is $v = cr$; hence its tangential acceleration is

$$\frac{dv}{dt} = \frac{cdr}{dt} = \frac{bcnr_2{}^2 \sin \theta}{r} = bcr_2 \sin \phi, \qquad (3)$$

since

$$r^2 = r_2{}^2 + n^2r_2{}^2 - 2nr_2{}^2 \cos \theta$$

and $\dfrac{d\theta}{dt} = b$, and $\dfrac{nr_2}{r} \sin \theta = \sin \phi$.

Therefore, the accelerating force is $Mbcr_2 \sin \phi$. and its component in the direction CO is

$$F' = - Mbcr_2 \sin^2 \phi. \qquad (4)$$

This being a push and not a pull is negative always, acting as a retarding force when θ is between π and 2π.

The rate of deviation of the mass is from equation (2),

$$\frac{v^2}{\rho} = c^2r - acr_1 \cos \phi = c^2r - bcr_2 \cos \phi, \qquad (5)$$

since $ar_1 = br_2$ from equation (1); so that the deviating force is

$$M(c^2r - bcr_2 \cos \phi).$$

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* When I speak of the force compelling the mass to move on a curve, I properly call it a "deviating" force; were I speaking of the action of P upon its controller I should call it the "centrifugal" force.

This force acts in the direction PI, and its component in the direction CO is

$$F'' = + M(c^2r - bcr_2 \cos\phi) \cos\phi. \tag{6}$$

Hence, the resultant tension along CO is

$$F = F' + F'' = M(c^2r \cos\phi - bcr_2) \tag{7}$$

$$= M(c^2r_2 - bcr_2 - c^2nr_2 \cos\theta),$$

since

$$r \cos\phi = r_2 - nr_2 \cos\theta,$$

from the figure; but

$$c^2r_2 - bcr_2 = a^2(r_1 + r_2)$$

by equation (1). Hence,

$$F = Ma^2(r_1 + r_2) - Mc^2nr_2 \cos\theta. \tag{8}$$

The first term is the constant deviating force which would be required to make the mass M, if concentrated at C, revolve about O with the angular velocity a and a radius $r_1 + r_2$. This term is always positive, as it means a constant tension on the connecting bar OC.

The second term is the component of the deviating force which would be required to make the mass M revolve about a stationary axis with a radius nr_2 and an angular velocity c, the resultant angular velocity of the mass in space. The essential sign of this term depends on the sign of cos θ. When

$$\theta > \frac{\pi}{2} \text{ and } < \frac{3\pi}{2}$$

the second term is positive.

It thus appears that the proposition I started to test is true for the case of simultaneous rotation about parallel axes. The resultant tension in the bar OC varies from

$$F\ max. = M[a^2(r_1 + r_2) + c^2nr_2] \text{ when } \theta = \pi \tag{9}$$

to

$$F\ min. = M[a^2(r_1 + r_2) - c^2nr_2] \text{ when } \theta = 0, \text{ or } \theta = 2\pi. \tag{10}$$

The value of F is zero when

$$c^2nr_2 \cos\theta = a^2(r_1 + r_2) \tag{11}$$

or

$$n = \frac{r_2 \sec\theta}{r_1 + r_2}, \text{ which becomes } \frac{CP}{CO} \text{ if } \theta = 0.$$

If

$$n > \frac{CP}{CO} \text{ and } \theta = 0 \text{ or } 2\pi,$$

there is compression in the bar OC.

The maximum tension in OC when $n = 1$ is

$$Ma^2 \frac{r_1^2 + 3r_1r_2 + 2r_2^2}{r_2}, \tag{12}$$

which occurs when $\theta = \pi$; and the maximum compression occurs when $n = 1$ and $\theta = 0$, viz.:

$$Ma^2 \frac{r_1^2 + r_1r_2}{r_2}. \tag{13}$$

If $n = 1$, the mass is on the circumference of rolling circle, and the path is an epicycloid, and

$$F_1 = M[a^2(r_1 + r_2) - c^2r_2 \cos \theta]. \tag{14}$$

If also $n = 1$ and $r_1 = r_2$ the path is a cardioid, and (12) becomes *max. tension* $= 6Ma^2r_1$ and (13) becomes *max. compression* $=$ $2\ Ma^2r_1$.

If the circle rolls along a straight line, $a = 0$ and $r_1 = \infty$ but as $ar_1 = br_2$, a finite quantity, $a^2(r_1 + r_2) = 0$; and we have a trochoidal path, with $c = b = \omega$ and

$$F_2 = -M\omega^2nr_2 \cos \theta.$$

If the plane of the circle is vertical, and it rolls on a horizontal track, the upward action of the track is

$$F = W + \frac{W}{g}\omega^2nr_2 \cos \theta,$$

which is identical with the result I obtained directly from the trochoidal motion.

Editors reprinting articles from this JOURNAL are requested to credit the author, the JOURNAL OF THE ASSOCIATION, and the Society before which such articles were read.

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TIMBER TUNNELING IN QUICKSAND.

BY RUFUS K. PORTER, MEMBER OF THE SANITARY SECTION OF THE BOSTON SOCIETY OF CIVIL ENGINEERS.

[Read before the Sanitary Section of the Society, March 1, 1905.]

THE city of Newton has lately finished the construction of a short sewer, about 500 ft. of which was built in timbered tunnels. These tunnels were constructed by the regular city laborers, good men, but with little or no experience in tunnel work. The ground penetrated was mostly quicksand, the two lower feet being wholly of that material. The level of the ground water was from 9 to 10 ft. above sewer grade. Trouble was feared on this account, but it was found as the work advanced that each section of tunnel was drained somewhat by the preceding sections, so that the head of ground water did not prove so troublesome as was anticipated.

The method employed in timbering the tunnels is described in detail, for in such books upon tunneling methods as have come to the attention of the writer, the different steps in the work which seem to him of importance have invariably been left out.

In the first place it was necessary to cut through the sheeting which formed the end of the tunnel shaft. A line of holes 4 ft. 6 in. long (Fig. 1) was bored through the plank 4 ft. above grade, and 18 in. below this another line of holes was made. As each plank was pierced by the second line of holes, the piece between the two lines was chiseled out and an inch board, 18 in. long, was set in and secured by wedges. Where there were braces

across the shaft which interfered with the legs of the first cap, an opening was cut across them large enough for the legs to be inserted. Any weakening of these braces was taken care of by bracing of a temporary nature, the details of which differed according to the place. A method sometimes used is shown in Fig. 2.

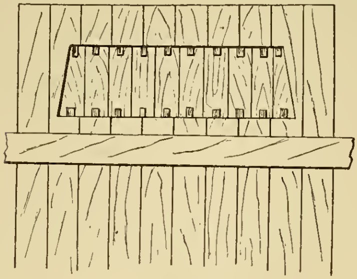

FIG. 1.

The first frame, consisting of a cap and legs, was set up against the bulkhead. The dimensions of these timbers are given in Fig. 3. The lengths of the legs, however, were altered from time to time, as necessary, the idea being to get the founda-

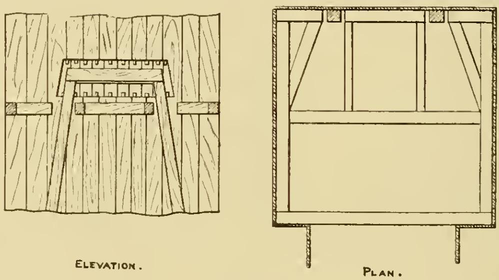

FIG. 2.

tions as low as possible. The ground was so bad that we were never able to see the footing of the legs, and even in the shaft

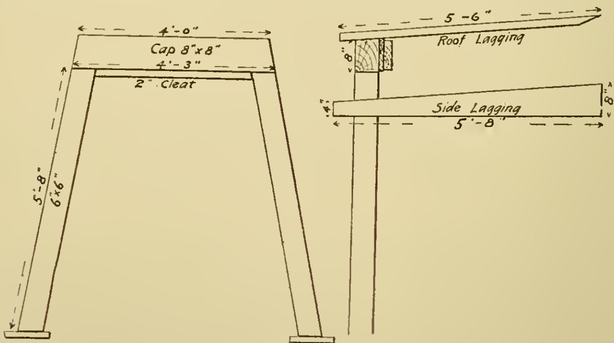

FIG. 3.

from which we started the sand boiled so seriously that boxes had to be used in which to set the legs.

An inch and a two-inch piece were nailed to the front of the cap, bearing against the breast, the cap proper being 3 in. from it, so placed that should the breast kick in a little at the bottom, there would still be some play.

The roof lagging pieces were then set up, the front end resting upon the cap, and the back end passing under and wedged down from a heavy timber running across the shaft, and braced or wedged down from the rangers. (See Fig. 4.) This wedging will be spoken of hereafter as tailing. The breast boards shown in the first sketch were then dropped a couple of inches, and the roof lagging was driven into the loose dirt behind the sheeting. After the roof lagging had been driven in far enough to stop any running of the material behind the sheeting, the breast boards were removed entirely and the sand behind them was allowed to run until it had assumed its natural slope. The two upper side lagging pieces on each side were then entered, being tailed in precisely the same manner as the roof. This side lagging was always driven along with the roof lagging. As the roof was given a pitch of about an inch and a half to the foot, it will be seen that had the side lagging been made rectangular and driven horizontally, a space would have been left between the roof and the top of the side lagging. To remedy this the first side-lagging pieces were made wider at one end than at the other, — and in wet places enough of these squegee or taper side-lagging pieces were used to give the sides a distinct downward inclination.

FIG. 4.

After the roof timber had been driven in about 3 ft., the sand was partially excavated, and a temporary cap or horse-head was set up (see Fig. 5), the dotted line showing the extent of the excavation to the point described. First the dirt under the roof was cut out and the cap was shoved up, until it rested upon the dirt. Then a small trench was cut under the middle of this cap and a center prop was set in, after which the side legs were set in place and tightened up with wedges. This work had to be done very cautiously on account of the nature of the ground.

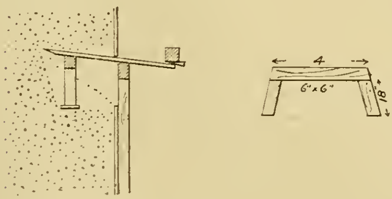

FIG. 5.

The horse-head being in place, the roof was then landed, that is, driven to the full length of the lagging. Of course, during all of these operations only enough dirt was removed to allow for the driving, for setting the horse-head, etc. Fig. 6 gives some idea of the excavation up to this time. The ground was then carefully cut away under one roof-lagging at a time, and a breast board (that is, an inch board 18 in. long) was set up perpendicularly under it. These boards were generally set on a footing board 2 in. wide and secured at the top by a wedge and thus helped the horse-head somewhat in supporting the roof. Any cracks in the lagging were stuffed with hay, which was found to be the best material for the purpose, because it allowed the water to drain through freely, while at the same time it held the sand back securely.

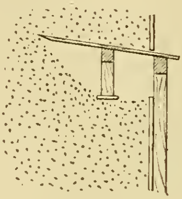

Fig. 6.

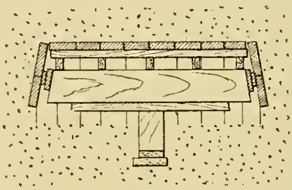

Fig. 7.

The cap was then brought in and set up against the breast boards under the end of the lagging, and adjusted to line and grade. (See Fig. 7.)

The cap was graded with a carpenter's level from the adjoining cap, the elevations of the caps being checked every two or three days by the grade party.

On top of this cap were placed five "chocking blocks," 7 in. long, 2 in. wide, and 3 in. thick. On top of these blocks was set a waling board 2 in. thick and of the same length as the cap. In driving, the side lagging followed the roof lagging, the first strip being about a foot behind the roof and the next one 18 in., and so on. The cap having been adjusted the two top side-lagging pieces were driven up flush with the breast and a wedge was driven between the cap and the side lagging, thus securing everything firmly in place. A narrow trench was next excavated under the middle of the cap, and a short center prop, resting on a foot block, set in. Between the foot block and center prop the point of a wedge was inserted and the wedge driven home, thus raising the waling board so that it bore firmly against the roof and supported it. The roof now being supported by

this waling board, or rather by the cap and the waling board, the horse-head could be taken out at any time that it interfered with the work.

An extra side-lagging strip was then driven on each side, and the dirt on each side of the center prop was carefully cut away, leaving an 8-in. perpendicular face. Against this face were placed two planks each 8 in. wide. The back ends of these planks butted together behind the center prop, and were secured at the sides by wedges driven between their opposite ends and the side lagging.

Two 8-in. by 6-in. timbers 10 ft. long, called "the bars," were then brought in, and the ends placed under the cap, butting against the breast boards just set. The middle of these timbers rested on legs with a solid foundation previously prepared, and the back ends passed under, and for the first section were wedged

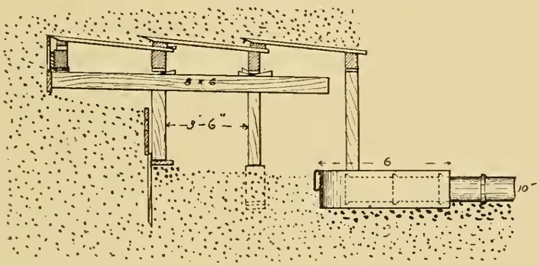

Fig. 8.

down from a timber placed across the shaft below a set of rangers. In subsequent sections they were wedged down from a cap already in place. Fig. 8 shows the arrangement of these bars. The sketch shows also the breast boards which were put in first, and what are called the halved breast-plank, the ends being supported by cantilever bars.

The center prop was then removed (the roof being supported by the bars) and the whole width of the breast was cut down at once. A side lagging on each side was first driven and the dirt cut away to a perpendicular face from 4 in. to 6 in., depending upon the amount of water in the ground. A plank of the same width was then put against this face and secured at each end by wedges. This work had to be done very quickly, as the material was generally too wet to hold a perpendicular face for more than a couple of minutes.

When the breast had been worked down in this manner as low as possible, inch boards, called cribbing boards, were set

upright against the breast and driven into the sand. These boards were made as long as could be driven, but seldom more than $2\frac{1}{2}$ ft. long, as it was rarely possible to work the breast low enough to set up longer ones, for it was necessary to work the breast down $2\frac{1}{2}$ ft. in order to set up $2\frac{1}{2}$ ft. cribbing. Sometimes it was impossible to get in more than two horizontal planks because of the softness of the breast. In that case the sand at lower edge of the last plank was leveled and a row of shingle was placed against it and forced into the sand. A shingle being 16 in. long, the bottom could be held by the sand and the top tacked to the cross plank of the breast.

When the sand was taken away a plank was immediately placed against the perpendicular face. The shingles being thus held by this plank, enough sand could be cut away to put in another breast-plank, after which another row of shingles was put in and the operation repeated. (See Fig. 9.) Shingles were used because they were most available, but were much of that work to be done $\frac{3}{16}$ in. iron plates, made about 16 in. long and 4 in. wide, would be found more satisfactory.

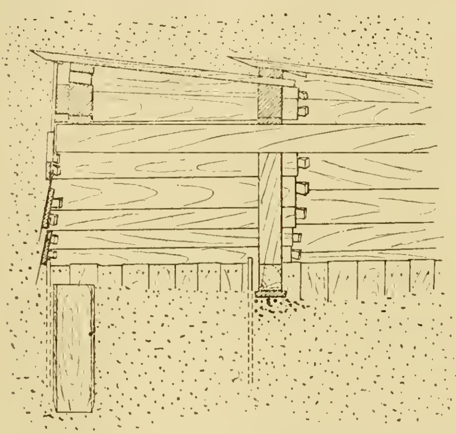

Fig. 9.

As it was unsafe to rely upon the bars for the support of the roof for any length of time, a long center prop was then inserted under the middle of the cap. It will be seen that it was necessary to put the foundation of this center prop as low as possible, to prevent undermining it when digging for the side legs; moreover, great care had to be taken in digging near the bottom of the cribbing to prevent the kicking in of the material at this point. Boxes were, therefore, constructed of inch boards, 2 to $2\frac{1}{2}$ ft. long, open at top and bottom, and 8 in. wide in the clear. These boxes were set upright on the sand where the footing of the leg was to be placed, and driven down with a sledge. As the box was sunk the sand inside of it was bailed out with a tin dipper, — each box being in effect a miniature caisson; when at the required depth, a foot block 8 in. square was placed

in it, the center prop was placed upon it and set up under the cap. The center prop was set low enough for the insertion of two wedges, between the cap and prop, which being driven home brought the weight of the cap up on the center prop.

The bars could then be taken out, and another side-lagging piece be driven on each side. After digging the ground out as low as possible, the side cribbing, which had to be below the underdrain, was driven and boxes were set up and driven for the side legs in the same manner as has been described for the center leg. These boxes were always driven lower than the bottom of the underdrain, but it was sometimes impossible to excavate more than halfway down inside of them on account of the boiling of the sand. In such cases the legs were a little short, and when it was found possible to hammer them down, an extra foot block was inserted, and care had to be taken to prevent kicking in at the top. The boxes used were a make-shift, improvised on the spur of the moment. Sheet steel boxes, running in grooves, so one side could be driven at a time, would be more efficient, since they could be driven down farther and more easily and could be made larger so that larger foot blocks could be used.

In order not to interfere with the drainage, 6 by 6 pieces were placed on each side of the section dug, close to the sides, and across these planks were laid, so that the sand coming out of the next heading would fall on the platform thus made and not obstruct the drainage underneath. In this way any sand that was dry, was kept dry and removed from the tunnel in that state. By the time this platform was placed, the section was generally very soft, but by the time that the material was ready for removal from under the platform it was generally found to be pretty solid, in part on account of drainage and in part on account of gravel put in.

DISCUSSION.

Mr. Fuller. — May I ask how this ground water was disposed of?

Mr. Porter. — It went through the 10-inch subdrain to a well sunk in the shaft, from which it was discharged by a pulsometer pump.

Mr. Fuller. — Didn't you find some clay washing into it?

Mr. Porter. — Yes. The subdrain was carried along with the tunnel, but kept three or four sections back from the heading to allow time for all the legs to settle to their load.

At intervals of 12 ft. through the tunnel the top of the drain pipe was cut off, and every once in a while a man would take the top off, shovel out the sand, and then replace the top. The regular cleaning rod was also used. The sand was very fine, but as there was quite a flow of water the pipe readily cleared itself, except when the bottom was being worked over.

Mr. Fuller. — Did you wrap your underdrain with burlap?

Mr. Porter. — No, we wrapped it with hay, which seemed to be a good medium, as it let the water in and kept the sand back.

Mr. Fuller. — How deep was this sewer below the natural surface?

Mr. Porter. — It varied in depth, but I should think the average depth was about 40 ft.

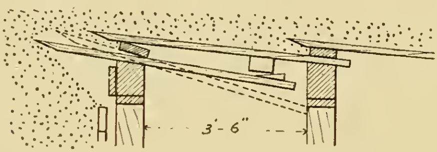

Fig. 10a.

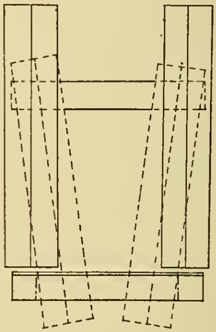

Fig. 10.

Mr. Fuller. — Was it sandy material all the way to the surface?

Mr. Porter. — It was all sandy material, but not all quicksand. In most of these sections the quicksand extended about halfway to the top of the tunnel. The bottom was invariably quicksand, and it is a curious fact that the tunneling was a great deal easier when the whole height of the tunnel was in quicksand. Fine quicksand acted like clay and would stay in place perhaps three minutes without support before it would fall in, and this time was found sufficient for the insertion of supporting planks.

In starting the next heading the roof lagging were entered between the waling board and the cap, the chocking blocks being chiseled out one at a time. (Fig 10a.) When the roof was started it was tailed, that is, held down by the cap; afterwards it was tailed down from the previous roof. As will be seen from the sketch, this left something of a void which must

be filled by settlement. It could be overcome by driving the whole lagging on the same slope, but that would necessitate, if the caps were to be level, a much thicker waling board, which would necessitate longer breast boards, and it was found that breast boards longer than 18 in. could not be used to advantage.

Mr. Fuller. — The breast boards were horizontal, were they not?

Mr. Porter. — The first breast boards were put in vertical before the caps were set. The others were all put horizontally across, and the cribbing at the bottom vertical, because it was driven into the soft sand. With those exceptions on the roof, I think all the rest of the work was just a repetition of what has already been described.

After the second section was finished, an extra cap and set of legs were inserted in the first section to help support the weight, which was thought to be too great for one set of legs.

In the worst places these sections settled slowly, that is, an inch in three or four days, until the second set of legs was inserted, when the rate of settlement decreased and finally stopped altogether. But considering the character of the ground there was very little settlement.

Mr. Fuller. — Will you show how the roof planks were driven? It looks as if the roof caps would interfere with the driving.

Mr. Porter. — But little weight was felt until the horse-head was in. By prying with the bar and pushing it in, and then changing the fulcrum of the bar and pushing it in again, the work was done. From the time that the horse-head was in until the roof was landed it was sometimes pretty hard pounding.

Mr. Fuller. — The roof planks were not tongued and grooved, I suppose?

Mr. Porter. — No; we trusted altogether to calking with hay; in fact, the roof was always dry, except where the tunnel was wholly in quicksand, when there was a slow dribble of water and sand which was stopped by calking with hay.

All of the drain pipes were laid in a scow and before setting this scow the bottom was sunk to the limits of safety. The scow used in Newton is simply a sheet of quarter-inch iron, bent as in Fig. 11. It looks something like a coffin. It is set over the pipe and the sand is then dug out.

A Member. — Were the pipes laid on the sand?

Mr. Porter. — It was found advantageous to put in a little gravel underneath.

A Member. — As a matter of fact, could you get the sand necessary for this purpose out?

Mr. Porter. — No, hardly. A man would stand in the box and dig for a pipe and then spend several minutes in getting his feet out so he could put the pipe in. Occasionally we could get the bottom low enough, though when the pipe went in the foundation material was half sand and water, and we dumped gravel all around the pipe and probably a great deal of this gravel was forced underneath the pipe. Sometimes, however, when the bottom was very bad, a miniature trench with sheeting was constructed in the bottom of the tunnel, and by this means the ground was lowered 6 in. or 8 in. lower than would have been possible otherwise. As a matter of fact, the pipe laying went a good deal better than it did in the open trench approaching

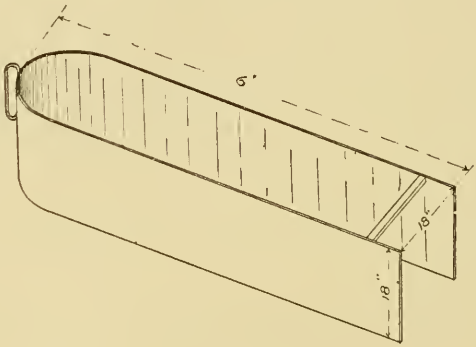

Fig. 11.

the tunnel, in the same sort of ground, on account of working from a face, and not tramping back and forth on the quicksand and thus making it soft.

It may seem as if we went through a great many operations, but, on account of the smallness of the gang employed, the work was not extravagant, and the progress, though slow, was steady.

I believe the maximum progress — working two shifts a day — was 48 ft. a week, and the average was perhaps about 20 ft. a week.

Mr. Fuller, — How long were those shifts?

Mr. Porter. — In the first tunnel we tried to work continuously, and we worked on 11-hr. shifts, but it was too hard on the men, and later we worked 10 hrs. The disadvantage of the latter arrangement was that there was just an hour

between shifts when the tunnel was standing with nobody in it. It seemed to be pretty hazardous quite often on account of the material. When you had 40 ft. of dirt on top of you and you went into this tunnel and saw the bottom shaking 20 ft. ahead of you, you felt somewhat frightened.

I have not said anything about the back filling. We could back fill about 8 ft. In back filling it was found easy to pack the material in except behind the cap; but a great deal of this tunneling was through vacant land, where a slight settlement on top would cause no serious damage. We found we could do very good work by taking the quicksand which had dried out on top, wetting it enough to make it plastic, molding it in balls and sticking it in as clay, and by then working it with a tamper.

Mr. Fuller. — Could any draining be done by driving pipes, like 2½-in. open-end pipes, through the breast horizontally?

Mr. Porter. — In the first section I drove two well points through the breast and into the sand, but the sand was so fine that it clogged the pipes. Of course, in the average quicksand you can bore wells and drain the sand out from underneath, if it is underlaid by a vein of gravel. I think Mr. Kimball took some such borings out in Newton, and the depth of the quicksand was something like 50 ft.

Mr. Kimball. — I went down as far as 100 ft. in one case.

Mr. Porter. — The dirt was taken out of these tunnels on a simple hand-car. We didn't bother to get any regular tracks, but used planks, and they seemed to work well. It was quicker to saw off a plank and put it down than to get rails together, besides being a great deal less expensive.

A Member. — What was the approximate cost of this work?

Mr. Porter. — Perhaps Mr. Farnham can tell better than I.

Mr. Farnham. — It was rather hard to determine the cost. The work was carried along with other work, but on a few sections where we had a chance to separate the cost, it amounted to about $16 a foot for completed work.

A Member. — That includes your pumping?

Mr. Farnham. — Yes; the pumping expense was comparatively small.

Mr. Porter. — Perhaps I ought to say that the gang comprised altogether five men and a foreman in the daytime, and four men and a foreman at night.

A Member. — Your foreman was a man who had had some experience before?

Mr. Porter. — He had never seen a tunnel before, but he was a good foreman, — a good trench man.

Mr. Farnham. — Mr. Porter was the man of experience.

Mr. Porter. — I think my experience was rather small. I would rather have a green man who uses his brains a little, than one who thinks that the only way to do a thing is the way that he has seen it done time and time again. We did not object to a green man, if he was a good man and used his brains to get over difficulties.

Mr. Fuller. — Was this tunnel completed before any of the sewer was laid?

Mr. Porter. — Yes.

Mr. Fuller. — Was there more than one of these tunnels?

Mr. Porter. — Yes; there were quite a number of them. The ground is very rolling, and when we came to high ground we generally tunneled it. One tunnel ran under the Sudbury River aqueduct, and at the request of the Metropolitan Water Board, we put in a tunnel there under compressed air.

Mr. Fuller. — You completed all your excavation before you put in the masonry?

Mr. Porter. — Yes; except in the air tunnel.

Mr. Metcalf. — Did you find any difficulty from settlement?

Mr. Porter. — Quite a little. Generally the cap settled before we got the legs under. I don't think we had more than a 6-in. settlement after we got a leg under. Some tunnels stood two or three months, and on one or two such occasions the caps went down a foot before we got the legs under.

A Member. — Did you have any trouble by the tunnels settling too much to get your work in?

Mr. Porter. — No; it was a pretty snug fit in one case, but we managed to get it all in.

Mr. Fuller. — How much of this timber was removed?

Mr. Porter. — Only the intermediate caps, but a great deal of timber was used that was useless for anything else. After the masonry was completed, when the underdrain was plugged and the pump was stopped, the water level rose and the water came through in small leaks; we never had a leak through the concrete invert, but we had some through little pin-holes in the brick arch.

Mr. Palmer. — What can you say about the leakage when you left the tunnel?

Mr. Porter. — It seemed to stop itself after a couple of weeks. The sewer is 20 in. by 30 in., and the bottom is only

6 in. wide, yet there was but three quarters of an inch of water flowing through it, which comprised all of the leakage through 1 500 ft. of sewer.

A MEMBER. — How much did you have to pump during construction?

MR. PORTER. — A 4-in. pulsometer pump was kept in operation continuously; there was not a full head of steam on, but it was a good flow.

A MEMBER. — What did you use for back filling?

MR. PORTER. — Quicksand. As I said, we moistened it enough to make it into balls. Of course, clay in tunnels is the best thing, but I found this quicksand worked satisfactorily. It was very peculiar sand, and acted a great deal like clay.

MR. FULLER. — On what sort of a foundation was the concrete invert laid?

MR. PORTER. — It rested on the ground over the underdrain, around which, and more or less over the entire bottom, gravel was put to bind it up. There is no sign of any crack in the sewer.

MR. FULLER. — What was the proportion of the concrete used?

MR. PORTER. — Well, I generally made it a little rich. I think it was about 1 : 2 : 4½. It is supposed to be 1 : 2 : 5, but we reduced the quantity of gravel a little because we wanted a little excess mortar.

MR. FULLER. — What was the size of the coarsest of the gravel?

MR. PORTER. — It was ordinary screened gravel; I don't think the coarsest particles were over an inch and a half in diameter.

MR. H. P. EDDY. — I notice that in place of the wedges used under the legs, as we call them, we have used at Worcester center legs to support the temporary roof block, instead of the wedges, as has been shown to-night. We have used screw jacks to hold the cap piece up tight.

The use of hay, which was spoken of, I have found to be of great service. I think that there is nothing better than hay. It is the miner's best friend.

In dealing with the underdrains, Mr. Porter did not speak of any method of keeping them free from obstruction except by digging out. We have frequently found it worth while to put a piece of rope in the underdrain, and to pull it back and forth, — a very old device. A tunnel that passes through

quicksand at the level of the floor and gravel at the roof, presents, I think, one of the most interesting and perplexing conditions to be met with, especially if the roof timbers have to be driven through gravel that contains bowlders or coarse stone. I don't know how many times I have tried to drive my roof timbers through bowlders and have then had to dig the bowlder out before the roof timbers could be inserted, and it is only with the greatest difficulty that the gravel can be prevented from running, under such circumstances.

It is also interesting at times to have a plane of coarse gravel a few inches below the bottom of the tunnel. This vein of coarse gravel is generally water bearing and the water being released causes the sand on top of it to boil badly, though the gravel stratum may, under some conditions, drain the fine sand overhead.

I think the use of shingles on the bulkhead is very interesting and of considerable importance in that kind of tunnel, and to me it is a new method. I don't remember seeing any account of that method, though it may have been explained before.

Mr. Farnham. — One thing has been presented to-night that is entirely new to me. Mr. Porter said that he was able to make quicksand stand about three minutes. If you will observe his methods, he was always careful to have something back of the material he was going to excavate before he excavated it. I didn't know that he ever let any quicksand stand more than three seconds because that material is so treacherous. Of course, I am speaking of material above the bottom. Most of the bottom was covered over by a floor; but the boiling was here taken care of by the planks driven at the sides far below the bottom of excavation. Other than that, the material was nearly always, and I supposed always, supported by something before the excavation was made. His shingle method was an ingenious method for that purpose.

On account of some questions that have been asked to-night it seems to me that all the members are not familiar with the general problem we had; that was spoken of previously by Mr. Kimball at one of the informal meetings of the Boston Society. Mr. Porter has confined himself entirely to the timber tunnel work, but it might not be out of place to speak a few words in regard to what our problems there were. The greater portion of this sewer was constructed through private land, across one street, and under two aqueducts of the Metropolitan Water Board. The soundings showed that there was fine sand the

entire distance at sewer grade. The topography of the country was of very peculiar formation, what we call "pot hole" formation. In laying out the location of the sewer we took advantage of these pot holes, and whenever we came to one of them we made an open cut. We started with an open cut at the Cochituate Aqueduct, and after that we tunneled a short piece to a point where the cut was about 25 ft., and there we again attempted an open cut. It was rather a novel experience for us, because we had a great deal more trouble with the open cut in the quicksand than we had with the tunnel. The material would slide in spite of all we could do. Mr. Porter called the street commissioner and myself up one day and we found that his timbers were all cracked, and the material had started to slide on the side a distance of 40 ft. away from the sewer. It looked rather shaky; with heavier timber and extra bracing Mr. Porter was able to hold it; but as soon as he got through that hollow and came to the next hill he started the tunnel. We were not sure how successful this method would be but we found that the tunnel work was more economical than the open-cut work, and we concluded that it would be better to tunnel than to work in open cut if the depth of sewer was more than 25 ft.

We also had in connection with this same piece of sewer a piece of pneumatic tunnel work which Mr. Porter has mentioned. That was carried on very successfully at the rate of 10 ft. a day. It was a more expensive method, but was made necessary on account of the crossing of the Sudbury Aqueduct, at which point it was absolutely necessary that there be no settlement whatever.

With this other work, this timber tunnel, we attempted, to find out how much settlement there was. A part of it was done during the winter and we took elevations upon the ground over the tunnel. So far as we could ascertain from elevations taken later, there was little or no settlement. There was a slight variation in levels taken, but nothing large enough or regular enough so we could be sure of the amount of the settlement. Probably there was a slight settlement at the surface, but nothing to speak of. There may have been a few voids left, but they were very small, and as the material gradually crawled, I think that in the depth of 40 ft. these voids were taken care of by the loosening up of the material without any apparent settlement on the surface.

The Chairman. — I think you gave the cost of the tunnel as about $16 a foot?

Mr. Farnham. — I should have said that nothing was included for shafts in that price. That was the cost of driving

the tunnel and building the brickwork in the short section the cost of which we were able to separate. The average cost with the shaft was considerably more than that. We had but two shafts; most of the openings were from short sections of open cut between the tunnels, and it was entirely impracticable to separate the cost of these as carried along.

THE CHAIRMAN. — I think you said the tunneling was somewhat cheaper than the open cut. Have you an approximate idea of the cost of the open cut?

MR. FARNHAM. — I have no figures to give that I could depend on. It was considerably more, nearly double, the cost of tunnel work.

THE CHAIRMAN. — I suppose you did not have a shaft?

MR. FARNHAM. — Only short pieces of open cut, except in one place where two shafts were sunk. These shafts were quite expensive. They were about 16 ft. square and the cost of sinking to a depth of 40 ft. was about $1,200 a shaft. That work was another example of the difficulties of open-trench work. We got down easily nearly to grade when the fines and began to boil in. This boiling sand, under a water head of about 10 ft., gave us a great deal of trouble. In the last shaft Mr. Porter sank, he had to put in a smaller center shaft at the bottom.

THE CHAIRMAN. — Did you encounter bowlders or coarse sand?

MR. PORTER. — In one place I did. The roof hit the top of a bowlder, but it was just the top and I dug under it, and as the bowlder settled an inch at a time, I kept the roof going against it, so I don't think I lost anything. We had got it out before we had to cut down for a breast.

MR. FRENCH. — This material was fine?

MR. FARNHAM. — Some of it was. It varied. We saved some samples which Mr. Kimball has tested. Some that he tried on the sieves went through the 100-mesh.

MR. KIMBALL. — There was one sample which passed entirely through the 200-mesh sieve.

MR. FARNHAM. — I learned from research of literature on tunneling that the proper way to drive a timber tunnel through quicksand was to first drive a small tunnel for drainage, then enlarge it, but there was little to tell how to drive the small tunnel. One practical suggestion I gathered from the literature on the subject was the use of hay. This was found to be of great assistance in the work. Mr. Porter used the hay constantly, and it worked very satisfactorily.

ENGINEERING ETHICS AND FEES.

By Frank C. Osborn, Member of the Civil Engineers' Club of Cleveland.

[Read before the Club, October 10, 1905.]

It was the intention to include in this paper a review of what has been accomplished in the field of ethics in other professions in this and other countries, but a lack of time has prevented this.

In our own country the subject has been before the American Society of Civil Engineers on two occasions and was quite thoroughly and even warmly discussed on both of them. In January, 1893, a resolution was presented asking for the appointment of a special committee " to collect information and to consider the propriety of the adoption by the society of a code of ethics for the profession, and to make such other recommendation as the committee thinks proper."

The Board of Direction presented to the business meeting of the annual convention a series of arguments for the adoption of the proposed resolutions as well as the arguments against it. These arguments are as follows (see vol. xlix, page 49, Trans. Am. Soc. C. E., December, 1902):

"Arguments for the Adoption of the Proposed Resolution.

" 1. The resolution is for the appointment of a special committee of this society, to collect information and to consider the propriety of the adoption of a code of ethics, and to make such other recommendation as the committee thinks proper. This does not commit the society to the adoption of a code of ethics, but simply proposes the appointment of a committee on this general subject. There should be no objection to the appointment of a carefully selected committee to consider the subject, and the members of the society will certainly have larger information and be in a better position to determine as to the propriety or impropriety of the adoption of a code of ethics after the report of such a committee than without such report, the subject never having been carefully considered or presented as a society matter.

" 2. The difference of opinion on this general subject which evidently exists among engineers is a good reason for the appointment of the proposed committee, so that the subject may be presented for further consideration and discussion in a clear and definite shape.

"3. There seems to be some question as to whether civil engineering may properly be called a profession. A clear statement of what the word means is given by the 'Century Dictionary,' which defines a profession to be:

"'The calling or occupation which one professes to understand and to follow; vocation; specifically, a vocation in which a professed knowledge of some department of science or learning is used by its practical application to affairs of others, either in advising, guiding or teaching them, or in serving their interest or welfare in the practice of an art founded in it.' The same authority adds: 'Formerly, theology, law and medicine were specifically known as *the professions;* but as the applications of science and learning are extended to other departments of affairs, other vocations also receive the name. The word implies professed attainments in special knowledge, as distinguished from mere skill; a practical dealing with affairs as distinguished from mere study or investigation; and an application of such knowledge to uses for others, as a vocation, as distinguished from its pursuit for one's own purposes. In professions strictly so called, a preliminary examination as to qualifications is usually demanded by law or usage, and a license or other official authority founded thereon required.'

"This definition seems very clearly to apply to the actual facts of the practice of civil engineering. Men who have any right to be called civil engineers, and who have any right to be corporate members of this society, do have a vocation in which a professed knowledge of some department of science or learning is used in practical application to the affairs of others. If the proposed committee finds it desirable it would doubtless recommend some means of carrying out or of enforcing regulations for such of the relations of engineers to their clients and to each other as could at all be made subject to rules. At all events, it is desirable that the committee should study this subject and report to the society.

"4. Rules do exist which govern the relations, to each other and to the public, of men following other professions or vocations in which there are associations of the members of such profession or vocation in some respects similar to this society. This applies, not only to the old so-called professions, but to other vocations. The dealers in stocks are subject to clear and definite rules, formulated and enforced through their exchanges. The dealers in produce are also subject to rules through their exchanges, and this is the case in other vocations.

"5. It will be well to appoint a committee to consider whether the dignity of civil engineering and the relations of members to themselves and to the public might not be improved by the application of rules of ethics or of conduct to be adopted by this society, as a representative body.

"6. It seems to be a fact that civil engineers do permit themselves in some few instances to act in ways which in other professions or vocations would be generally considered improper

and unprofessional acts. Such acts are contrary to the rules of etiquette which have become standard in those professions. The appointment of a committee to see whether this can in any way be remedied as regards civil engineers is desirable.

"7. The lack of rules by which departures from the proper line and professional line of conduct can be judged affords excuses for unprofessional conduct when such conduct is based upon ignorance of, or indifference to, professional morals. The adoption of rules and principles governing the relations of engineers to each other and to their clients would restrain the unprincipled and guide the ignorant.

"8. The fact that many engineers are engaged in salaried positions should be an argument in favor of rather than against, the appointment of a committee to consider the possibility of a code which would add to the dignity of the profession. Those who require the services of engineers are apt to base their appreciation and estimate the value of those services upon the standing which the engineer himself takes. It is inconsistent with the function of this society to depreciate engineering. It is its duty to endeavor to elevate it in every relation, and the appointment of the proposed committee will be a step in this direction.

"ARGUMENTS AGAINST.

"It is held that one may just as well assert that the other great professions are great in spite of their codes as because of them. Neither assertion can be proved. It seems to be a fact, however, that the lawyers have no formal code. Their rules of conduct are partly matters of tradition and partly of interpretation and construction, by various authoritative bodies, and form a valuable mass of professional ethics, but they have not been codified and adopted by the great bar associations in the United States.

"Furthermore, one of the most important medical societies of the United States, the Medical Society of the State of New York, has voted to abandon its code of ethics. The fundamental reason for this action is summed up in these words of the president of that society: 'It would comport more with the dignity of the medical profession, and would enhance the respect in which it is held by the general public, if all specific rules of ethical conduct were elided from the by-laws of the State Medical Society, and if the regulation of such matters were hereafter left to the judgment of individual practitioners, influenced by the well-known consensus of professional opinion and by local custom.'

"It is believed that the architects also are without a formulated and accepted code of ethics. So we see that the professions are either without codes or are beginning to abandon them. The rules governing the transactions of the stock and produce exchanges are not analogous to what is understood by a code of professional ethics; they provide simply for the carry-

ing out of specific contracts, many of which are not enforcible by law, but the carrying out of which is vital to the existence of the business of the exchanges.

"It is held that it is impossible to frame a code of ethics that would cover all cases. Such a code would be too complicated and minute to be successfully administered. The code, therefore, must be merely a statement of general principles; but such statements have been made by moral teachers ever since society began, and the general principles which govern human conduct are sufficiently well formulated already.

"A strict code would restrain those who need no restraint, but it would not be regarded by the unprincipled further than their own interests dictated. It would happen sometimes, perhaps often, that an engineer would be deterred from doing his duty to his client or to himself by a timid obedience to the code, or that he would make the code an excuse for not doing disagreeable things which reflected upon the honor or capacity of other engineers.

"It is held that specific cases of violation of the traditions and spirit of the profession should be treated individually, each on its merits, and that this can be done, so far as the American Society of Civil Engineers is concerned, by presenting the case to the Board of Direction, or that a special body or committee can be created in the society to perform this special function."

A study of the arguments seems to show that those against the resolution are much easier to answer than those in favor of it; however, the Board of Direction recommended to the society that no such committee be appointed.

The question then discussed was whether the society should issue a letter ballot on the appointment of a committee in accordance with the resolution. During the discussion it was stated that the conduct of the members "could fairly be left to their intelligence and intention to do right"; that codes of ethics can "scarcely be found in written form"; that "we do not see them in other associations"; that "a code of ethics must be formulated upon the good sense, good breeding and general comity of man to man."

The vote taken at this time resulted in a majority against the resolution in the proportion of about 5 to 1, although only 65 votes were cast.

At the annual convention in 1902, one of the subjects for discussion was: "Should Engineering Practice be Regulated by a Code of Ethics?" In opening this discussion Mr. Geo. A. Soper said (see vol. xlix, page 46, Trans. Am. Soc. C. E., December, 1902):

"There is nothing new or formidable about the idea of a

code of ethics, any more than there is about a list of by-laws of which, in the case of the society, the code might very possibly become a part. Moral codes were old in the time of Moses, and have been indispensable ever since. George Washington had a reference guide of action, and Franklin, a chart of conduct. In a broad sense, the Constitution of the Union is a code of ethics, as are the various public laws and ordinances under which we live.

"In a narrower sense, and the one in which we must use it here, ethics is the subject which treats of those moral acts which, though they cannot make us liable to bodily restraint, are capable of exercising a powerful influence for the good or ill of our fellow man.

"It would be the object of a code of ethics to express the united voice of the society upon such points as the following:

"1. To what extent and in what ways may an engineer advertise his services?

"2. Under what circumstances is the owning and exploitation of patents consistent with a strictly professional practice?

"3. What general principles should govern the conduct of consulting engineers toward other engineers engaged upon work?

"4. Under what circumstances should an engineer pass judgment on the work of his professional brethren?

"5. To what extent should the circumstances of an engineer's private life be allowed to interfere with his professional standing?

"6. Is it practicable or advisable that the society should attempt to expose quacks?

"7. What acts, if any, should make an engineer liable to the censure of the society?

"If we consider that every man is a man of principle, that is, has definite ideas as to the moral consequences of his acts, and decides upon a doubtful course of action only after referring the matter to his conscience, we will see that he has a code of ethics, or moral standards, whether or not they happen to be written down. In the same way, this society, as an aggregation of moral and responsible men, has an unwritten code of engineering etiquette. The main principles are known to all. The finer points, the nicer discriminations, are chiefly in the keeping of the older members. The majority of the young men are constantly improving their professional sensibility through experience, and rectifying their courses of conduct by the moral pattern set by the great chiefs. Here and there, there are ex-

ceptions; men who appear indifferent to their status, or, like raw recruits, are unable to discern, without help, what is expected of them.

"The argument for a written code is the argument that the moral teachings of the pillars of the profession should be crystallized into such form as to be available and accessible to all, down to the last recruit. The argument against expresses the opinion that the recruit should be left to find out for himself what is good and what is bad for him and his fellows.

"As the one who has been asked to open this discussion, the speaker cannot announce himself as a warm partisan of either side. He has felt the need of knowledge of the ethics of the profession, and has set out to get it after the usual fashion. He has been an observer of courses of action on the part of some which he thought should not pass without the censure of a united engineering fraternity, and he has seen worthy members of another profession advance through a coöperation and support of their fellows which would be impossible with us under present circumstances.

"If there is need of a written code of ethics in the American Society of Civil Engineers, that need should make itself felt naturally and spontaneously. A code, based on less than a practically unanimous vote of approval, would be difficult to introduce, and, in the end, might prove more of an incumbrance than a help. But, backed by a solid sentiment, a code should be a benefit."

Mr. Benjamin M. Harrod, past-president of the American Society of Civil Engineers, remarked that "the principles of professional ethics should be explained and impressed in engineering schools, and a full and clear discussion of them should form part of the transactions of engineers' societies. In this way an unwritten law will prevail, with all the authority, and more completeness and flexibility than can be given to a written code."

Mr. J. J. R. Croes, past-president of the American Society of Civil Engineers, says that the question, "Should engineering practice be regulated by a code of ethics?" must be answered in the affirmative, and further that "no business, trade or profession can be successfully carried on in any community except in accordance with the ethical standards which prevail in that community. . . . Among what we consider the civilized nations of the present day, the standard of ethics is, as has been well said, embodied in the Ten Commandments of

the Mosaic Dispensation and the Golden Rule of the Christian Dispensation." Mr. Croes says further, however, that the formulation and establishment of such a code, under existing circumstances, by means of a written law of action applicable to civil engineers alone, is impracticable; that an acceptance of the principles of such a code would imply a liability to some kind of penalty for infringement, and this would require the establishment of a court of arbitration or adjudication, and such a code or such a court could not be established by a society including only a small proportion of the total number of practitioners.

Mr. Croes closes his discussion of the subject by saying: "While the promulgation of a formal code of ethics for the civil engineer is not practicable or desirable, there is doubtless a need and a demand for a public setting forth of some of the elementary principles which have come to be recognized by engineers of experience . . . as fundamental, but which the young practitioner, . . . fresh from his college halls, . . . has no familiarity with, and of which the ordinary man of business, unfamiliar with professional ethics, has little conception. . . . It would be an advantage, therefore, if some civil engineer of acknowledged experience and standing would write a brief compendium of engineering ethics, from his point of view, and have it published by a standard bookseller. It would promote discussion and lead to much good. . . . Any effort to give to such a publication the status of an established law of morals would be absurd."

Mr. W. Hildenbrand suggested a short and simple code comprised in a single sentence: "Act as a gentleman, in every respect, on all occasions; be just and generous to your brothers in the profession, and do not hesitate to take the full responsibility for all your actions!"

Mr. Hildenbrand argues for the establishing of a sort of court or tribunal to investigate and deal with transgressors, and closes his remarks as follows: "The object of this discussion proves that the need for rectifying or bettering the ethics in the engineering profession has been felt, and the writer believes that such a need cannot be better or more efficaciously supplied than by a body of truth-loving men known as 'The Court of Justice of the American Society of Civil Engineers!'"

The American Institute of Architects has put practically this idea into its constitution, a portion of Section 3, Article VII, reading as follows: "The Board of Directors shall annually elect a Judiciary Committee from its own membership and

establish rules for its guidance. This committee shall hear and adjudge all complaints of members against members, and its findings shall be conclusive upon all questions of fact involved in the evidence submitted. Appeal in writing may be made to the Board of Directors upon questions of professional or ethical policy."

The Institute has, however, never published a code of ethics, although recognizing the existence of an unwritten code by providing for infractions of it.

The Boston Society of Architects has adopted a code of ethics which is published in a former handbook of the Cleveland Architectural Club, recommended by the club to its members. This code is as follows:

"SECTION 1. No member should enter into partnership, in any form or degree, with any builder, contractor or manufacturer.

"SECT. 2. A member having any ownership in any building material, device or invention, proposed to be used on work for which he is architect, should inform his employer of the fact of such ownership.

"SECT. 3. No member should be a party to a building contract except as 'owner.'

"SECT. 4. No member should guarantee an estimate or contract by personal bond.

"SECT. 5. It is unprofessional to offer drawings or other services 'on approval' and without adequate pecuniary compensation.

"SECT. 6. It is unprofessional to advertise in any other way than by a notice giving name, address, profession and office hours, and special branch (if such) of practice.

"SECT. 7. It is unprofessional to make alterations of a building designed by another architect, within ten years of its completion without ascertaining that the owner refuses to employ the original designer, or, in event of the property having changed hands, without due notice to the said designer.

"SECT. 8. It is unprofessional to attempt to supplant an architect after definite steps have been taken toward his employment.

"SECT. 9. It is unprofessional for a member to criticise in the public prints the professional conduct or work of another architect except over his own name or under the authority of a professional journal.

"SECT. 10. It is unprofessional to furnish designs in competition for private work or for public work, unless for proper compensation, and unless a competent professional adviser is employed to draw up the 'conditions' and assist in the award.

"SECT. 11. No member should submit drawings except as an original contributor in any duly-instituted competition, or

attempt to secure any work for which such a competition remains undecided.

"SECT. 12. The American Institute of Architects' 'schedule of charges' represents minimum rates for full, faithful and competent service. It is the duty of every architect to charge higher rates whenever the demand for his services will justify the increase, rather than to accept work to which he cannot give proper personal attention.

"SECT. 13. No member shall compete in amount of commission, or offer to work for less than another, in order to secure the work.

"SECT. 14. It is unprofessional to enter into competition with or to consult with an architect who has been dishonorably expelled from the 'Institute' or 'Society.'

"SECT. 15. The assumption of the title of 'architect' should be held to mean that the bearer has the professional knowledge and natural ability needed for the proper invention, illustration and supervision of all building operations which he may undertake.

"SECT. 16. A member should so conduct his practice as to forward the cause of professional education and render all possible help to juniors, draftsmen and students."

The *Engineering News* in November, 1892, published an editorial which is well worth a careful reading.

It states that a code of ethics is a matter of slow growth, a gradual general acceptance of rules of conduct that have been more or less traditional, and that "the only aim of such a code is to establish equity and courtesy between different members of the same profession, and whoever distinctly violates equity and courtesy in his treatment of a fellow engineer, violates — if not an already established code, as we believe he does — at least what will be the code when there is one, and what every engineer should seek to establish."

It states that some few engineers, at times, presume "on this absence of a rigid code to violate with expected impunity the most elementary requirements of courtesy and fair dealing in pursuit of their own selfish ends. This ought not to be so, but it is."

This editorial defines one principle of a code of ethics as follows:

"1. It should be considered unprofessional and dishonorable for any engineer to accept a call in consultation for any work which is already in charge of an engineer, *except the call come from or through such engineer in charge.* Engagements tendered from principals should be considered or accepted *only* when there is no engineer already in charge of the work." The editorial

goes on to remark that "justice, courtesy and expediency alike demand that the letter as well as the spirit of the rule should be most rigidly respected."

If engineers show a proper and dignified respect for each other they will command that same respect from others; and *vice versa*, if they do not show a proper regard for each other they cannot expect a very high opinion from the public.

A second principle is set forth as follows:

"2. It should be considered unprofessional and dishonorable for any engineer to report upon any work which is already in charge of an engineer *except to such engineer in charge.* Reports to principals direct should only be made when there is no engineer in charge; nor then, when there has been an engineer in charge whose professional acts are impeached in such report, without prior tender of a copy of such parts of the report as may personally concern him, for his perusal and response."

Both of these principles seem axiomatic and yet it seems to have been considered necessary, or at least advisable, to enunciate them.

A crystallization of the opinions of American engineers seems to indicate that there should be a code, that there is an unwritten code that engineers should be familiar with, but that the time for promulgating a complete and written definite code with suitable provision for penalizing transgressors has not arrived.

The Canadian Society of Civil Engineers thought ten years ago that a written code was desirable, and by something more than a two-thirds vote adopted such a code, making it a by-law of the society. This code is printed in full in *Engineering News* of February, 1896.

The question of fees for engineers is one that has not received the attention it deserves. Very little has been said in engineering societies or in the engineering press on the subject of standard charges for engineering services.

One reason for this is that a comparatively small proportion of the engineers has been engaged in independent practice. This proportion is increasing, however, and the need is growing for something in the way of standard charges for engineering services, and it would seem that a thorough discussion of the subject would be justifiable and beneficial.

A complete schedule for all branches of an engineer's work would be naturally difficult to arrive at; but for certain classes of work a fee based on a percentage of the cost of the construction

seems practicable and equitable. The architects have had for years such a system, and it has grown to be recognized by the public and by the courts. A few engineers have adopted the architects' schedule, and if more of them would do so the result would be of advantage to all concerned.

It applies naturally to bridge work and to manufacturing plants, and probably would be found applicable to other lines of work, although when a large proportion of complicated mechanical work is involved, the usual amount of the fee, *viz.*, 5 per cent., is hardly sufficient. In fact, in many cases a fee of 10 per cent. is not too much to cover complete plans, specifications and superintendence.

For surveys for electric and steam railways, preliminary work, the percentage system is not so applicable, and the plan of charging per mile seems more practicable.

For some classes of large work a plan of payment has been adopted by which the client pays the actual book cost of the engineering work plus a fixed sum per month to represent the profit.

This plan is probably a fair one for both engineer and client, but does not lend itself readily to general adoption for all cases, as the monthly profit should vary with the magnitude of the work involved.

Is it not a duty to ourselves as well as to the public to take up this subject, discuss it thoroughly and arrive, if possible, at some basis for engineering fees that will be fair and equitable?

OBITUARY.

Charles Henry Wellman.

MEMBER OF CIVIL ENGINEERS' CLUB OF CLEVELAND.

CHARLES HENRY WELLMAN was born in Nashua, N. H., in 1863. He was the son of a New England iron master, Samuel Knowlton Wellman, so that the eminent position he achieved in the metallurgical world may be justly regarded as a striking illustration of the influence of heredity and environment. Having only the advantages of a common and high-school education, he had the great advantage of a capacity for continued hard work and study, and he also had the faculty of attracting to himself men who could assist and supplement him in his work. His kindly and sincere nature drew around him a large circle of friends.

In 1881 he entered the employment of the Morgan Engineering Company, of Alliance, Ohio, where he served his apprenticeship as machinist and draftsman until 1885, leaving there to become chief draftsman of the Otis Steel Company, of Cleveland, Ohio. From this position he was transferred to the Open-Hearth Department of that company as superintendent. This was the beginning of his connection with the open-hearth industry, towards the advancement of which he contributed so large a share. In 1890 he was appointed superintendent of the Wellman Steel Company at Thurlow, Pa., and occupied that position until 1895, when he went to South Chicago to finish up the construction and take charge of the operation of the Illinois Steel Company's Open-Hearth Department, and remained there until 1896, when, in company with S. T. Wellman and J. W. Seaver, he founded the Wellman-Seaver Engineering Company of Cleveland, Ohio, which is now known as the Wellman-Seaver-Morgan Company. In this company he filled the position of chief engineer and general manager, and while occupying these positions he did the work that gave him the world-wide reputation as an engineer and constructor that he enjoyed when he died. In 1897 he organized the Electric Controller and Supply Company, of Cleveland, Ohio, of which he was president until the day of his death.

In 1888, he was married to Miss Bertha Adams, of Cleveland, Ohio, and their home life was an ideal one in every respect. Their beautiful home, on the shore of Lake Erie near Cleveland, was

most happily named "Open Hearth," and was the center of a wide circle of friends, particularly noted in the world of music. His wife, two sons and a daughter survive him. His death was caused by the railroad wreck at Mentor on the night of June 21, 1905.

In his death the community at large, the various manufacturing industries with which he was prominently connected and to whose advancements he contributed so much, the old Stone Church of which he was a faithful and consistent member, this club and the various other clubs and engineering societies of which he was a valuable member, have suffered a great loss. In conclusion, the words of one of the hundreds of messages of condolence received after his death may be quoted:

"Few men have done so much and left so many friends."

Editors reprinting articles from this Journal are requested to credit the author, the Journal of the Association, and the Society before which such articles were read.

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THE LEVEE AND DRAINAGE PROBLEM OF THE AMERICAN BOTTOMS.

By Edwin G. Helm, Member of the Engineers' Club of St. Louis.

[Read before the Club April 5, 1905.]

As the title of this paper indicates, it is meant to discuss the protection from overflow by the water of the Mississippi River and the drainage of the interior waters of the American Bottoms; and when this expression "American Bottoms" is used, it is meant to include that portion of the bottom land of the Mississippi Valley lying between the river and the foot of the bluffs through the counties of Madison and St. Clair in the state of Illinois. This same name is often applied to all that portion of the bottom lands in the state of Illinois south of St. Clair County as far as the Ohio River; but in all that follows only the local application of that term is meant, and particularly that part north of Prairie Du Pont village.

This district extends from the city of Alton, about 9 miles north of the limits of the city of St. Louis, southward past the entire length of the latter city to a point almost opposite its southern limit. The line separating the counties of Madison and St. Clair extends due east through this district from a point about opposite Dock Street in the city of St. Louis, so that this area extends in Madison County about 16 miles north of the dividing line and in St. Clair County about 7 miles south of it. The average width from bluffs to river in both counties is about 7 miles; but at the northern end the bluffs come practically up to the river bank at the city of Alton, and at the

southern end the bluffs approach within about 3 miles of the river. This district is divided into three general drainage areas, with the following-named streams as the principal outlets: Wood River at the extreme north, Cahokia Creek near the center and Prairie Du Pont Creek at the south. These are the only streams of importance that flow into the Mississippi River through the American Bottoms.

Wood River is formed by the confluence of two branches, the East and West Fork, which come together at just about the point where they may be said to come out of the bluffs; it flows southward a distance of about 3 miles to the Mississippi River. The two forks of Wood River have their sources in the southern part of Macaupin County about 16 miles above its mouth. This stream has a drainage area in the bluffs of about 117 sq. miles and in the bottoms of about 3 sq. miles, and has a maximum discharge of about 2,900 cu. ft. per second. The channel in the bottoms lies entirely west of the Big Four Railroad, and except in extreme freshets its flood waters are confined west of that railroad. On June 29, 1902, the water from this stream was 6 and 8 ft. deep in the streets of East Alton, washed out the tracks of the Big Four, and overflowed that part of the bottoms just east of the Big Four as far south as Edwardsville Crossing, where it was prevented from going any further south by the embankments of the Illinois Terminal Railway (excepting what passed through a 12 in. or 15 in. pipe). This has happened a few times before, with less extreme height. Generally speaking, Wood River may be considered as having no effect on the American Bottoms except upon that portion contained in its own drainage area west of the Big Four Railway about 3 sq. miles.

The Cahokia Creek drainage area comprises over half of the total area of these American Bottoms, or it may be generally said, all that portion north of the Vandalia Railroad and east of the Big Four. This stream has its source in the vicinity of Litchfield in Montgomery County, flows in a general southwesterly direction, entering the bottoms about 12 miles north of the south line of Madison County, nearly 35 miles south of its source. In the bluffs it drains an area of 228 sq. miles; at about the point of entering the bottoms it is joined by Indian Creek, a tributary with a drainage area of about 38 sq. miles, so it may be said that Cahokia Creek enters the bottoms with a drainage area behind it of 266 sq. miles, or a discharge of about 5,040 cu. ft. per second. This drainage area is more than double

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the entire area of the American Bottoms. After entering the bottoms, Cahokia Creek flows nearly south to about the county line, then nearly west, passes through the city of East St. Louis and empties into the river near the southern limits of that city. Its straight line length in the bottoms is about 20 miles, but owing to its tortuous course its actual length is probably 40 or 50 per cent greater. In the bottoms, it has an almost uniform grade of 1.5 ft. to the mile.

Prairie Du Pont Creek on the south has its source in the bluffs in the southern portion of St. Clair County, or rather it is formed by the confluence of several smaller streams, which all together drain about 42 sq. miles in the bluffs of the southwestern part of St. Clair County, the longest of these tributaries having its source about 10 miles from the foot of the bluffs. The point where Prairie Du Pont Creek enters the bottoms is about eight miles south of the Baltimore & Ohio Railroad, which is the northern boundary of that portion of the American Bottoms which naturally drains south into this creek. From this point the Creek flows westward about 3 miles to the village of Prairie Du Pont and thence southward along the Mississippi River a distance of about 5 miles before emptying into it.

In all surveys and investigations made by the writer, the datum plane used has been the low-water mark or zero of the Mississippi River gage in the city of St. Louis, and all elevations given in this paper are with reference to that datum.

The variation of the general level of the ground in the American Bottoms is from about elevation 18 to elevation 60. Generally speaking, the variation in the level in any mile or two does not exceed 20 ft. from the lowest to the highest points. The general slope of the Mississippi Valley from the Missouri to the Ohio is about half a foot to the mile, and this general average holds throughout this stretch of the American Bottoms. This general grade is varied, however, by the secondary slopes following the limits of the different drainage areas. Going more into details, we have first the case of Wood River, a stream coming out of the bluffs at a point only a short distance from the Mississippi River and having a solid rock bed for a distance of nearly a mile after entering the bottoms, which will permit of little or no further erosion, so that the grade is held up, so to speak, as its bottom through this rock gorge is relatively nearly 10 ft. higher than other similar streams with earth bottoms. Consequently, when the rock bottom is passed, the stream rapidly falls this distance of 10 ft., and having been

given a great velocity by this rapid descent, it continues to scour and cut out so that when it finally enters the Mississippi River, its bottom is quite as low as the low-water mark of the Mississippi River at that point, or relatively to the river, about $7\frac{3}{4}$ feet lower than any stream south of Wood River in the American Bottoms, the bottoms of both Cahokia and Prairie Du Pont creeks being about 7 ft. above the low-water mark of the Mississippi River at their outlets. From Wood River the land with an elevation of about 55 gradually slopes upward in a southwestern direction and reaches a summit elevation of about 60 in about 1 mile. This ridge extends southwesterly about 8 miles and is an effective divide between the Wood River and Cahokia drainage areas. In a direct southerly direction, however, the land has a general down-grade from Wood River between the Big Four Railroad and the ridge above mentioned and through what is known as Grassy Lake. This latter region has an elevation of about 45 near East Alton, gradually widens southward, and at a point about 8 miles south of East Alton it stretches nearly from the Big Four Railroad east to the bluffs, and forms a part of the Cahokia Creek Bottoms, having an elevation at this point of about 40. Cahokia Creek enters the bottoms about 2.5 miles above this point, where the average height of the ground along the creek is about 50, so that down to this point the Cahokia Bottoms have a fall of about 3 ft. to the mile, while the creek itself has a fall of 1.5 feet to the mile. From this point of junction of the Grassy Lake district and the Cahokia Bottoms they form one continuous bottom extending from the bluff westward nearly to the Big Four Railroad and extending southward nearly to the county line, a distance of 8 miles. Throughout the entire bottoms, Cahokia Creek has a nearly uniform fall of 1.5 ft. to the mile and the bottom lands down to the county line have practically the same average fall, the Cahokia Bottoms having an elevation of about 26 near the county line. Just north of the county line, however, there is an immense basin known as Horseshoe Lake, comprising about 3.5 sq. miles, whose bottom is practically the same as that of Cahokia Creek at the southern end of the lake, or elevation 20. This lake is never dry, it having 18 in. to 2 ft. of water in the dryest seasons. It is connected directly with Cahokia Creek at its southern end, but the overflow water from Cahokia also flows into it from the northeast through what is known as Elm Slough in times of heavy rain. This lake bed is the lowest body of land in the American Bottoms, not only lower with reference

to the grade of the river directly opposite, but actually as low as any land south of it in these American Bottoms. These are two points that must always be considered; for actual elevation is not so important as the elevation relatively to the grade of the river. From Cahokia Creek near the county line southward the entire bottoms from the bluffs to East St. Louis rise rapidly, and within the first mile they rise to an average elevation of about 38, or fully 18 ft. higher than the bottom of Horseshoe Lake. A ridge or divide is thus formed between the drainage areas of Cahokia and Prairie Du Pont creeks, and from here on the land has a fairly uniform fall of 1 ft. in 6 000 ft., or about half what the fall is in the Cahokia Bottoms in Madison County. This fall continues southward to Prairie Du Pont Creek, and there again the land rapidly rises in less than half a mile after crossing the creek to an elevation nearly as great as the ground just south of Cahokia Creek. It then falls gradually, going south to the Fish Lake district in the southern end of St. Clair County.

To sum up, we find that the bottoms from East Alton to Prairie Du Pont are divided into two separate and distinct and altogether different districts; one, the Cahokia Creek district, comprising practically all bottom lands in Madison County, and the other, the Prairie Du Pont district, comprising practically all the bottoms in St. Clair County north of Prairie Du Pont. The Cahokia district has a natural drainage channel, Cahokia Creek running practically through its entire length, and the grade of the land and of the creek is 1.5 ft. to the mile, or three times the grade of the river, while the Prairie Du Pont district has no natural drainage channel at all and the grade of the ground is only about one-half what it is in the Cahokia Creek district, being only slightly greater than the fall of the river itself. As a result of this, the Prairie Du Pont district comprises a series of lakes of various sizes caused by the water from the bluffs flowing on and being unable to flow away; on account of the slight fall of the land and the absence of any well-defined drainage channel, it gradually spreads out and practically forms lakes on what is relatively high ground. Generally speaking, the low lands in St. Clair County, while they are actually about the same elevation as those in Madison County, are relatively from 5 to 8 ft. higher when compared with the grade of the river directly opposite. For example, Horseshoe Lake is at elevation 20, but is 16 ft. above the low water of the river, directly opposite, whereas Big Lake in St. Clair County is at elevation 22, but

is 24 ft. above low water of the river opposite, or relatively 8 ft. higher than Horseshoe Lake.

Now let us compare these two districts relatively to the amount and disposition of the water falling upon or flowing through them. The Cahokia Creek district, in addition to the water coming from the creek proper, receives the water of the Madison County ditch, which drains the Grassy Lake district above mentioned, also of Judy's Branch, Schoolhouse Branch, Cantine Creek and Caseyville Creek, and a few other minor drains flowing off from the bluffs. In the Prairie Du Pont district we have only two streams of any consequence coming out of the bluffs north of Prairie Du Pont Creek, namely, Schoenberger Creek and Brouillette Creek. The latter at present flows into Big Lake, which in high stages gradually drains southward into Prairie Du Pont Creek. Schoenberger Creek in times of heavy rains fills up the low lands and flows in both directions, south into Big Lake and north into Spring Lake. The dry weather flow, however, is mostly northward. It has no well-defined channel any longer in the bottoms, and changes its direction of flow almost at will. In the writer's opinion it should properly drain southward, as it did years ago; in what follows it will be classed with the Prairie Du Pont district.

Spring Lake above mentioned at present drains westward and then northward through the Baltimore & Ohio Railroad into the Cahokia Creek bottoms; so in reality Schoenberger Creek is at present more particularly draining into Cahokia Creek. Cantine and Caseyville creeks come out of the bluffs at just about the summit of the divide between the two drainage areas, and have always presented probably the most difficult problem for solution in the entire American Bottoms. On account of the extreme flatness of the country in this vicinity and the peculiar location of these streams, they often flow in either or both directions; and as they are heavy, silt-carrying streams, the deposits from them have changed their course and direction more than once. They both at one time flowed north, but now all of Caseyville Creek is flowing south, and much of the overflow water from Cantine Creek; so that as they both come out of the bluffs between the Vandalia and Baltimore & Ohio Railroads, and have filled up, from 2 to 8 ft., the land between these two roads and at the bridges where they flow through, they have been the occasion of no end of lawsuits and contentions between the land owners and these railroads. Although they flow south through the Baltimore & Ohio, they flow into

Spring Lake and then back through the Baltimore & Ohio again near East St. Louis into Cahokia Creek; so there is no question but that they should be classed with the Cahokia Creek drainage area.

DRAINAGE AREAS OF AMERICAN BOTTOMS.

	In Bottoms.			In Bluffs.			Total.	
	Area sq. m.	Length.	Discharge Cu. ft. per sec.	Area sq. m.	Length	Discharge Cu. ft. per sec.	Area.	Discharge Cu. ft. per sec.
Wood River	3	2	100	117	14	2 810	120	2 910
Indian Creek	5	2	160	33	14	1 050	38	1 210
Cahokia Creek N. of L. & M.	16	5	510	228	35	3 830	244	4 340
Madison Co. Ditch	15	8	480	..	..	..	15	480
Cahokia Creek S. of L. & M.	48	15	1 530	..	..	..	48	1 530
Judy's Creek	..	..	..	12	4	690	12	690
Schoolhouse Creek	5	2	160	8	4	510	13	670
Cantine Creek	3	2	100	22	9	910	25	1 010
Caseyville Creek	1	1	30	12	4	690	13	720
Schoenberger Creek	13	3	410	13	5	660	26	1 070
L. & N. to I. C.	10	..	320	..	..	..	10	320
Brouillette Creek	..	..	..	7	3	490	7	490
Goose Lake District	7	4	220	..	..	..	7	220
Cahokia Lake District	5	..	160	..	..	..	5	160
Prairie Du Pont Creek	..	..	..	42	10	1 200	42	1 200
Total for Cahokia N. of Wabash	..	..	..	..	..	..	266	5 040
Total for Cahokia N. of Judy's Br.	..	..	..	..	..	..	309	6 720
Grand total for Cahokia Creek	93	..	2 970	315	..	7 680	408	10 650
Total for Dist. No. 1 N. of Prairie Du P.	35	..	1 110	20	..	1 150	55	2 260
Grand total for Dist. No. 1	35	..	1 110	62	..	2 350	97	3 460
Grand total for Cahokia and Dist. No. 1.	..	..	..	..	..	..	505	14 110
Grand total for entire bottoms	..	..	..	..	..	..	625	17 020

The accompanying table gives the length, area and maximum discharge of all of these streams in each of the three drainage areas of the American Bottoms. These values of maximum discharge are calculated from the well-known Burkli-Ziegler formula for run-off, using values of 1.5 in. per hour as maximum rate of rainfall and coefficient of 0.33 for proportion of run-off. The writer has found this to check very closely with the greatest discharge ever experienced from some of these streams as calculated by Kutter's formula from the area and slope of the streams at maximum stage, of which very accurate measurements and data have been obtained by the writer in connection with a recent investigation and survey for the purpose of ascertaining the liability of the Litchfield & Madison Railroad in certain suits for alleged obstruction of the flow of the water of Cahokia Creek. From this table it will be seen that in the Cahokia Creek district the creek drains a total of 93 sq. miles in the bottoms and 315 sq. miles in the bluffs, or, in other words, the bottoms must receive and care for the drain-

age of three and one-half times their area in the bluffs, whereas in the Prairie Du Pont district north of Prairie Du Pont Creek this creek drains 35 sq. miles in the bottoms and only 20 sq. miles in the bluffs, or, in other words, the bottoms receive the drainage of less than two-thirds their area in the bluffs. This is, of course, exclusive of Prairie Du Pont Creek itself (the proper manner of comparison) as this creek does not in reality flow through the above area, but is at the extreme south end of it, and is merely the outlet, and does not overflow any part as Cahokia does. Even including Prairie Du Pont Creek, it makes the area drained in the bluffs less than twice as large as that in the bottoms and leaves the relative proportion of bluff area to the bottom area drained twice as large in the Cahokia district as in the Prairie Du Pont district. This is considered by the writer a very important point in the solution of this drainage problem, for it is not the water that falls upon the bottoms that causes most of the trouble, but the water that comes from the bluffs, and if this bluff water can be conducted through the bottoms to the river without damage, this drainage problem is practically solved.

Ordinarily, there are about 4 sq. miles in Madison County constantly under water, and in St. Clair County about 5.5 sq. miles. In Madison County the effect of a heavy rainfall in the Cahokia Creek watershed is to cause the stream to overflow its banks from the point where it enters the bottoms down to East St. Louis. Through the city of East St. Louis there has never been a rainfall that produced any serious consequences. It is only the back water from the river at a high stage that causes serious trouble to East St. Louis. The two highest floods in recent years in the Cahokia Bottoms were those of June, 1902, and June, 1904. The first of these was caused by the heaviest rainfall ever recorded by the weather bureau at St. Louis, 4.7 in. for 24 hours. The latter must have been caused by a local cloudburst in the upper watershed of the creek; for the weather bureau reports throughout central Illinois and at St. Louis have no indication of excessive rainfall. The height of these two floods was very nearly the same, that of 1904 being about 2 in. higher than that of 1902. The waters were confined within narrow bounds along the creek by the naturally high ground and a few small local levees till about due east of Mitchell they passed these confines that held them in up to that time, and the water then poured out over the entire bottoms and flowed west, north and south, and, augmented some-

what by a portion of the water which had broken through a small levee near Poag and also the overflow from Indian Creek, practically covered two-fifths of the entire drainage area of the Cahokia Creek in the bottoms, or about 38 sq. miles. When it is understood that a large portion of this land overflowed was comparatively high and never covered by high water from the river since 1844, the seriousness of the situation can be appreciated. Landowners are endeavoring to fix the blame for this on the various railroads crossing the area flooded, and so far suits aggregating several thousand dollars have been filed, with prospect of many more. All of this water flowed into Horseshoe Lake and the low bottom lands adjoining, with the result that the level of the lake was raised about 6 ft., or to elevation 32, as the lake had already about 6 ft. of water in it on account of the high stage of the river. This was the highest stage ever reached by Horseshoe Lake except when overflowed by the river. The writer has calculated from the height the water was raised and the area that the amount of water entering this lake was about 2,000,000,000 cu. ft., or a little less than half of the total water that fell in the entire watershed, showing that about half of it ran off. Horseshoe Lake acts as a storage reservoir or a sort of safety-valve for the country west of it; and, were it not for it, there is no doubt that East St. Louis would be flooded oftener from Cahokia Creek than it is from the river. As it is, the flood waters spread out over this lake, raise its level and then gradually flow off through the creek to the river. The effect of heavy rainfall is much less serious in the Prairie Du Pont district than in the Cahokia district. This is on account of the fact that there is much less volume of water to deal with, as explained above, also because numerous lakes act as reservoirs, as in the case of Horseshoe Lake, and allow the water to flow off gradually through the bottoms, and again because Prairie Du Pont Creek, which carries the largest volume of water, does not overflow the bottoms north at all, but passes its flood water on down stream and into the river.

In what has preceded reference has been made to the general north and south grades of the land, and little or nothing has been said as to the east and west grade. In Madison County there is, generally speaking, a slight up-grade from Cahokia Creek westward to about the vicinity of the Chicago & Alton Railroad, and from there west to the river there is a slight down-grade, which increases in rate as the river is approached; in other words, the Chicago & Alton Railroad may be said to have

been built along a sort of divide between the land draining directly west into the river and that draining east into Cahokia Creek; that is, the Chicago & Alton Railroad is located on the highest possible ridge consistent with good alignment between East St. Louis and East Alton. The only ground as high as this west of the Chicago & Alton is a short stretch along the Chicago, Peoria & St. Louis Railway north of Oldenberg and what is known as Ebenezer Ridge, running southwest from Mitchell. But between Oldenberg and Mitchell there is a low tract of country that gradually gets lower and spreads out westward to the river till it finally occupies the entire river front from the Madison County line northward. In St. Clair County the land is nearly level, east and west, but there is a slightly ascending grade from the bluffs westward to the East St. Louis & Carondelet Railroad, or Conologue, as it is popularly called. From the Conologue westward, however, there is a decided down-grade to the river, and in fact the land immediately on the east of that railroad is from 3 to 8 ft. higher than it is on the west side, showing that this railroad was built along the extreme western edge of the highest plateau south of East St. Louis, the natural location of a levee, for which it was originally built, as will be explained later.

In all that has preceded as to topography of the American Bottoms, reference is had, of course, to general average levels and grades. There are minor areas that are much higher or lower than is stated for the general condition. The whole bottom is more or less a series of ridges and slashes, but the general topographical features are as stated above.

The average level of the land in the American Bottoms for which use is made for agricultural or commercial purposes, is from 30 to 35 ft. above the low water of the Mississippi River, directly opposite. Anything below that is of little value or practical use unless it is artificially protected or drained. Land lying higher than 35 ft. above the low water of the river is, of course, proportionately more valuable, but is scarce. Probably not 10 per cent. of the American Bottoms is as high as that. As during the past sixty years the stage of the Mississippi River has been above elevation 30 on sixteen annual occasions, and has during the same period been practically at or above elevation 35 on seven occasions, it can be understood that the problem confronting this district is serious. The dates and heights of these seven extreme stages are as follows: 1844, 41.4 ft.; 1851, 36.6 ft.; 1855, 37.1 ft.; 1858, 37.2 ft.; 1883, 34.8 ft.;

1892, 36 ft.; 1903, 38 ft. The two stages of 1844 and 1903 stand out prominently as the highest of which there is any authentic record; and the 1844 stage is remarkable in that it is over three feet higher than the next highest recorded stage, while the successive differences between the five next highest recorded stages are not over 0.8 of a foot, showing that this stage of 41.4 was phenomenal; yet with storm conditions similar to those that prevailed at that time, it is quite possible to occur again, but may hardly ever be exceeded. This subject of high river stages and their causes is a study in itself and will not be treated further at this time. Suffice it to say the high water of 1844 practically covered the entire American Bottoms; there being no railroad embankments at that time to obstruct its flow, it passed in one continuous sheet over the entire area. That of 1903 would probably have covered nearly as large an area were it not for the various railroad embankments that have been constructed since 1844. As it was, the greater portion of all the land between the Southern Railroad and the Baltimore & Ohio Railroad in St. Clair County, and in Madison County all the land east of the Chicago & Alton Railroad and north of Litchfield & Madison Railroad and Long Lake was protected from the overflow of 1903.

In the past there have been four attempts made to give some relief from the conditions surrounding the American Bottoms. The first was about forty years ago, when a corporation was formed under the name of the American Bottoms Board of Improvement Company, which received a special charter from the state legislature empowering it to enter into contracts with land owners in the American Bottoms, to construct and maintain levees and make ditches and to collect from the land owners a stipulated annual amount for the protection or drainage thus afforded. As a result of this a levee was constructed, which is now the embankment of the Conologue Railroad; also some small ditches were made, of minor importance. In 1868 the Supreme Court of the state declared the charter under which this company was operating to be unconstitutional, which stopped all further work by this company. Subsequently the rights of this company to the levee were sold or leased to a man by the name of Conologue, who built a railroad upon it, and after subsequent transfers it finally passed into the hands of the Terminal Railroad Association of St. Louis, the present owners. Under the circumstances, the present status of this embankment and the obligations of the

present owners toward the district for levee protection are questionable. The high water of 1903 was barely over this embankment, which successfully withstood a stage of 36.3 ft. before it broke, but in 1892 it broke at a stage below 36 ft. The Terminal Railroad put it in good shape again last year, and it is probably better now than it ever was. It varies in height from 5 to 14 ft. and has side slopes of 1.5 to 1.

The second attempt made to correct the situation in the bottoms was in 1882, when all that portion of St. Clair County in the bottoms lying south of the Vandalia Railroad and east of Tenth Street in the city of East St. Louis, which was then the city limit, was organized into Drainage District No. 1 of St. Clair County. This was done under the statute which permits the organizing of drainage and levee districts upon petition of the majority of land owners, and the taxing of land proprietors in proportion to the benefit received. There are two statutes in Illinois under which this may be done, the chief difference between them being that under one the commissioners, three in number, are elected by the land owners, and under the other they are appointed by the county judge. That portion of this district south of Prairie Du Pont Creek was subsequently separated from it and formed into a district by itself, so that the present boundary of this district on the south is Prairie Du Pont Creek. This district constructed a main ditch from Big Lake south to Prairie Du Pont Creek, near where it comes out of the bluffs, and also one or two laterals, and a levee along the north side of Prairie Du Pont Creek from the Conologue to the bluffs. According to the original profiles, this main ditch had a fall of less than 2 ft. from the bottom of Big Lake to the bottom of Prairie Du Pont Creek, a distance of over 4 miles; in other words, a fall of less than half a foot a mile, or less than the Mississippi River. Needless to say it was not many years before this ditch was of little or no value, as very little cleaning or repairing was ever done upon it; and to-day, in place of having 2 ft. fall from Big Lake to Prairie Du Pont Creek, the bottom of the latter is slightly higher than the bottom of Big Lake, the ditch is filled up from 4 to 6 ft. and in consequence the depth of water in the lake is from 4 to 6 ft., with no probability of much lowering. For the past year the writer has been at work, under the commissioners of this district, investigating and endeavoring to work out a practical solution of these difficulties. This will be considered later.

The third attempt at relief was the formation, some time

during the 80's. of the Chouteau, Venice and Nameoki Levee and Drainage District, which comprises all that part of Madison County lying north of the Madison County levee and east of the American Bottoms levee along the river, and south and west of Long Lake and the Big Four Railroad, extending as far north as Oldenberg, and containing in all about 18 000 acres. This district constructed the north and south levee along the river from Oldenberg south to Venice, where connection was made with the Chicago & Alton Railroad, and also the east and west levee, extending from just south of Venice east to Stallings, along the greater portion of which is now built the St. Louis, Troy & Eastern Railroad. There was also built a cross levee extending from the Chicago & Alton Railroad at Mitchell southwest along the northern edge of the Ebenezer Ridge above referred to, to the north and south levee. All of these levees are 10 ft. wide on top and have side slopes of 2.5 to 1 on both sides. The west and the south levees are both built on very low ground, their heights varying from 10 ft. to 16 ft., but the cross levee is built on very high ground, its height varying from 1 to 8 ft. These levees at no point were more than a foot above the high water of 1903, and in many places the water was as high if not higher than the levee. All of the levees broke and afforded little or no protection in 1903, and also in 1892, with the exception of the cross levee, which has successfully withstood all high waters since it was built, and has been an important factor in the protection of the land south and west of it. That portion of the north and south levee between Oldenberg and the cross levee has broken repeatedly, even at a moderate stage of the river, and the course of this levee has been changed in places several times, but all of no avail. It is just opposite the mouth of the Missouri River, and this together with its great height and the fact that the top of it has never been much above high water make it very inefficient as a protection. All of these levees were constructed in an improper manner, as all the earth of which they are made was excavated from deep borrow pits on the river side of the levee so close to its toe that in many cases the slope of levee and that of the borrow pit formed practically one straight line, there being no berm at all. This is also one bad feature of the Conologue levee in St. Clair County, above referred to.

The fourth and last attempt to improve conditions was the organization several years ago of the Elm Slough Drainage District, which had for its purpose the construction of a new

channel for Cahokia Creek into Horseshoe Lake through Elm Slough. This ditch was very imperfectly made, its size being only about one-fourth that of Cahokia Creek, and nothing was done to prevent overflow of the land south, so that in a few years this new channel was practically filled up again, the creek followed its old channel, and the organization of this district practically passed out of existence.

As stated above, that part of St. Clair County south of Prairie Du Pont Creek was detached and formed into a separate district, which has constructed a fairly good levee along the southeast bank of Prairie Du Pont Creek and some other minor levees, which with improvements contemplated upon them bid fair to provide a very good protection of that district. It is not intended, however, that this paper shall discuss anything south of Prairie Du Pont. At the present time the only drainage and levee districts having any practical existence are the Chouteau, Venice and Nameoki District in Madison County and Drainage District No. 1 of St. Clair County.

In addition to the levees above mentioned, there have been three important railroad embankments which have played no small part in affording more or less protection to certain areas. These are the Chicago & Alton embankment in Madison County, and the Baltimore & Ohio and Illinois Central embankments in St. Clair County. The former was above the high water during 1903 through a large portion of its length, and formed a very substantial protection to parts of that country. The Baltimore & Ohio Railroad is the highest embankment in the entire American Bottoms, has a nearly uniform elevation of 45 from East St. Louis to the bluffs, and was fully 4 ft. above the high water of 1903, which it successfully withstood, absolutely protecting East St. Louis from being completely inundated. Since the flood it has been double-tracked for over half its length, so is much stronger than before. It would be all the protection that East St. Louis could desire on the north were it not for one bad feature, and this is a trestle opening 80 ft. in length through it at the head of Spring Lake, about 4 miles east of East St. Louis, which under present drainage conditions cannot be closed. There are about 1 000 acres of low lands north of the Baltimore & Ohio which naturally drain south through this opening in the Baltimore & Ohio, and then back again north through the flood gates near East St. Louis into Cahokia Creek; but the owners thereof are not in favor of draining straight north into Cahokia Creek, and thus permitting this trestle to be closed up; and so

far no practical diversion of this drainage has been worked out. The other railroad embankment, the Illinois Central, on the south, is about level with the 1903 high water for half its length, and the other half, next to the bluffs, is 4 ft. higher. This embankment broke during the last high water at a point near the middle of its lower portion and in consequence the south end of East St. Louis was flooded. Had it been as high on the west as it is on the east, there is little doubt that it would have held. Farmers and land owners have done more or less local leveeing and ditching along the streams through their lands throughout the American Bottoms, but it has all been of little consequence, and of purely local and partial benefit, if any. The above represents the present status of the entire levee and drainage matter in the American Bottoms.

Plans for Relief. — From time immemorial, it has generally been contended that the only proper solution of this whole problem was the formation of one large district, and some have even advocated including everything down as far as Chester, where the bluffs come up to the river again as at Alton, and constructing a large channel along the foot of the bluffs from Wood River, or at least from Cahokia Creek, through to the southern extremity of this big district at the Mississippi River, this channel to drain the entire country, both bluffs and bottoms. Then there was to be constructed a large levee along the river front from Alton to this southern extremity; and it was the idea of a levee " from bluffs to bluffs," as it was popularly described, that gave birth to the idea of going as far south as Chester. This whole plan certainly has a popular ring to it, and does not have to go far to secure friends or advocates; and the writer must admit that he himself was much taken with the idea of a channel for Cahokia Creek along the foot of the bluffs as far as the south line of St.Clair County when he first became interested in this subject, six or eight years ago, and did no little work along those lines; but more extended investigation in later years has thoroughly convinced him of the mistake in this idea. The fact that a small amount of water has, in times of heavy rains, overflowed from Wood River, passed through Grassy Lake and in a most roundabout manner finally reached Cahokia Creek, has been the origin probably of the idea of extending this large channel along the bluffs north to Wood River and bringing its water south through the American Bottoms. Following this same line of evolution, it was only a step further to form the stupendous plan of simply deepening this big channel a little

bit (as it is explained) and making it a ship canal through which the waters of the Mississippi River would be permitted to flow in times of high water, thus reducing the height of the river in the main channel, and through which large river steamboats and barges would be navigated, thus forming a line of water transportation through this interior country along which large factories would gradually spring up; and thus to develop these bottoms in a phenomenal manner. It sounds very grand and certainly is pleasant to think about, if it were only easy of accomplishment. Of course, in any improvement it is essential that the necessary funds be provided in some manner; and as the big scheme mentioned above would require millions, it is natural that the promoters of these plans should look first to the largest financial resource we have, namely, the National Government, and it has long been the dream of many that the national Congress might be induced to open the coffers at Washington and do these things for us. The recent stand taken by Congress, however, on the levee question in the South, and the general fate of the regular river and harbor appropriation bill during the past few years, has about put a quietus on the hope of getting the National Government to build our levees along the river for us; for it can scarcely be claimed that the construction of the levee to keep out a 44 high water would deepen the channel of the river at low water one particle, and there is no present danger of the river cutting a new channel along the east side at any time, and these are the only possible excuses for a government appropriation. There is still the ship canal; that is the government's principal business just at this time, so we should surely be able to induce it to construct our ship canal along the bluffs! The government, through the engineers of the Mississippi River Commission, has just completed an investigation and survey of the most feasible route for a waterway from Alton to St. Louis as a part of the proposed 14-foot channel from Chicago to St. Louis, and although the report on this is not yet made public, it is known that the location of this channel will lie between the Chicago, Peoria & St. Louis Railroad and the river, and that it will join the river just north of the Merchants Bridge. Why should this location be chosen in preference to all others, or at least in preference to the one along the bluffs? First, it is the straightest and shortest; second, it would cross no railroad tracks, whereas, along the bluffs, every railroad entering St. Louis from the east would be crossed (23 in number), which would mean either drawbridges or raising of tracks at least 50 ft.

above, and the railroad companies would never permit their tracks to be broken with drawbridges; third, there is a complete absence of any drainage trouble along the Chicago, Peoria & St. Louis, whereas to maintain the canal along the bluffs, it would first be necessary to construct an auxiliary channel to care for the drainage, so that in no sense could the canal be used for a drainage canal as its promoters so strongly advocate. Millions are to be spent at Panama to divert the flood waters; the same principle holds good in this case; fourth, any deep waterway that is ever constructed through the Mississippi Valley must improve and use the St. Louis Harbor; for it is not likely that the three St. Louis congressmen will permit the one representative from the American Bottoms to have money appropriated to remove St. Louis inland for all time to come; to have two deep-water channels so close together would be out of the question. The writer has estimated that along these lines it would cost at least $3 300 000 for excavation alone for a canal only 100 ft. wide in the bottom; and with the elevation of the 23 railroads and building of necessary locks, etc., the total cost would exceed $6 000 000. This amount, if applied to the main channel of the river, would increase its carrying capacity by at least three times what the carrying capacity of this canal would be, so as a relief channel this canal would certainly not be an economic success. This is not drainage at all, and has nothing to do with drainage.

The more conservative have advocated the large district plan and the construction of simply a drainage channel along the bluffs and a levee along the river front. To this end bills have been presented in the Illinois legislature at various times in the past, and there is one before it at this present session. The main feature of all these bills has been the organization of a large district, at first to reach from Alton to Chester, but from the determined opposition to anything of this kind always offered by the Representatives from Monroe County its advocates have of late practically limited all plans to Madison and St. Clair counties. For the purpose of raising money to carry on this work it is proposed to issue bonds and pay interest and principal by a tax on the assessed valuation of all the property within the district, the bill at present before the Legislature stating that this tax shall not exceed 2 per cent. per year on the assessed valuation. Now let us see what this means. The accompanying table gives the areas and actual values according to the assessors' books of all the land in the American Bottoms, in de-

		Madison County.												St. Clair County.				
		Wood River Township.	Chouteau Township.	Edwardsville Township.	Venice Township.	Nameoki Township, South of L. & M. R.R.	Nameoki Township, North of L. & M. R.R.	Collinsville Township, South of L. & M. R.R.	Collinsville Township, North of L. & M. R.R.	Granite City.	City of Venice.	Village of Madison.	Total for Madison County.	Centreville Sta. Township.	Stites Township.	City of East St. Louis.	Total for St. Clair County.	Grand Total for Both Counties.
Land west of old districts and west of proposed new district	Area acres	2 800	7 520		2 890						40	290	13 540	1 500			1 500	15 040
	Land val.	270 000	450 000		220 000						65 000	315 000	1 330 000	150 000			150 000	1 480 000
	R. R. val.	180 000	380 000		70 000						20 000	155 000	715 000	110 000			110 000	825 000
Land in old districts and west of proposed levee	Area		1 440		1 130		1 930			790			5 290					5 290
	Land val.		80 000		85 000		145 000			1 475 000			1 785 000					1 785 000
	R. R. val.		80 000		30 000		105 000			165 000			380 000					380 000
Land in old district and east of proposed levee	Area		320		480	1 280	5 940			900	900	800	10 620	21 610		3 590	25 200	35 820
	Land val.		20 000		35 000	95 000	445 000			1 680 000	1 445 000	1 005 000	4 725 000	2 160 000		15 000 000	17 160 000	21 885 000
	R. R. val.		15 000		15 000	70 000	325 000			180 000	505 000	300 000	1 420 000	1 620 000		2 680 000	4 300 000	5 720 000
Land in proposed new district outside of old districts	Area	3 840	9 920	15 150		12 070	12 080	6 080	640		210		59 990	4 800	1 300	1 840	7 940	67 930
	Land val.	340 000	600 000	1 360 000		905 000	905 000	455 000	50 000		340 000		4 955 000	480 000	145 000	13 030 000	13 655 000	18 610 000
	R. R. val.	240 000	490 000	610 000		665 000	665 000	245 000	25 000		115 000		3 055 000	360 000	110 000	2 900 000	3 310 000	6 425 000
Total land in American Bottoms	Area	6 640	19 200	15 150	4 500	13 350	19 950	6 080	640	1 690	1 150	1 090	89 440	27 910	1 300	5 430	34 640	124 080
	Land val.	610 000	1 150 000	1 360 000	340 000	1 000 000	1 495 000	455 000	50 000	3 155 000	1 850 000	1 320 000	12 785 000	2 790 000	145 000	28 030 000	30 965 000	43 750 000
	R. R. val,	390 000	965 000	610 000	115 000	735 000	1 095 000	245 000	25 000	415 000	640 000	455 000	5 630 000	2 090 000	110 000	5 580 000	7 780 000	13 410 000
Average valuation per acre of area	Land	95	60	90	75	75	75	75	75	1 865	1 605	1 255		100	110	5 150		
	Railroad	55	50	40	25	55	55	40	40	210	565	375		75	85	1 030		

tail for each subdivision, and also the values of railroads separate from the land valuation. The assessed valuation is one-fifth the actual value according to the laws of the state of Illinois, so that a 2 per cent. tax would be really two-fifths of 1 per cent. on the actual value. From these figures it will be seen that there is great irregularity in the valuations in the two counties of Madison and St. Clair. They show that in Madison County the railroad valuation is $5 000 000 and the land valuation is $12 000 000. In St. Clair County the railroad valuation is $7 000 000 and the land $30 000 000, and of this amount East St. Louis alone comprises 85 per cent. of the total, the figures for East St. Louis being $28 000 000 for the land and $5 000 000 for the railroads, and the total for the entire bottoms being $44 000 000 for the land and $13 000 000 for the railroads, or a grand total for railroads and land of $57 000 000. Now from this it will be seen that in all regular taxes and also in this levee and drainage tax, if it should be made on the basis of valuation as is contemplated, St. Clair County, with about one-fourth of the total area, would pay over two-thirds of the tax, and East St. Louis alone, with only about one twenty-fifth of the total area, would pay just about three-fifths of the tax. The railroads alone would pay about one-fourth of total tax and the farm lands in St. Clair County would pay about 50 per cent. more than the farm lands would in Madison County. As the lowest possible figure for carrying out these big levee and drainage plans is at least $2 000-000, and will perhaps run over $4 000 000, according to their extent, this means that East St. Louis alone would pay at least $1 250 000 and perhaps $2 750 000 and the railroads in the bottoms at least $500 000 and perhaps $1 000 000. If a bill of this kind is ever passed, and the matter comes before the people to be voted upon, it is not very likely that the citizens of East St. Louis will approve of the measure when they are made acquainted with the facts and understand that they are to be taxed for $1 000 000 for something that can be provided for them under existing laws and in a better and safer way for one-third that amount.

All these various schemes and plans are mentioned merely to show why we have not already had satisfactory arrangements made to drain and protect the American Bottoms. They distract attention from feasible and simple solutions of this problem under existing laws, as there is constantly the hope that the National Government will furnish all the funds, or, at least, that some one else beside the one benefited will pay for this improve-

ment. Nothing of this kind will ever be put through unless it is paid for by the landowners themselves and in proportion to the benefits received; and as soon as the landowners settle down to this, just so soon shall we get results.

From what precedes it will easily be understood how much greater interest the city of East St. Louis has to be protected from the Mississippi River overflow than all the remainder of the bottoms put together; so it is not surprising to see that city endeavoring to protect herself and provide a suitable sewerage and levee system of her own, independent of the remainder of the bottoms. To that end, in December, 1903, the people of that city voted on a proposition to tax themselves for levee purposes 1 per cent. on the assessed valuation for seven years, which would amount to about $60 000 per year, or $420 000 for the seven years, if it was found necessary to raise that amount. This was carried by a large majority, and the first $60 000 of this amount has been collected this spring. The endeavor has been to make arrangements with the Illinois Central Railroad to raise its tracks as above described and to do some local levee work along the east bank of the Cahokia Creek through the city and up to a connection on the north with the Baltimore & Ohio Railroad, which is already an excellent levee as stated above. This of course left unprotected the district between the creek and the river, known as the Island, where all the freight terminals are. It would be impossible to protect this district without changing the course of the creek. The negotiations in regard to the use of these railroad embankments dragged along without any satisfactory results, and finally it became very apparent to the writer, at that time city engineer, that these railroad companies were very much averse to any plan of this kind that did not contemplate the protection of practically all railroad terminals in the city. So the writer finally outlined the plan of raising and strengthening the Conologue Railroad, and constructing a strong levee along the river front through the property of the Wiggins Ferry Company and Southern Railroad Company up to a point just north of Brooklyn, and of constructing a new channel for Cahokia Creek east from this point through the west end of Horseshoe Lake and along the first section line north of the Madison County line to the existing Cahokia Creek channel; also of constructing a levee along the south side of this new channel from the proposed levee at the river, east to the bluffs. This would protect practically the whole of St. Clair County and part of Madison.

The writer submitted this plan to the management of the Terminal companies, asking that a commission be appointed to investigate the matter and endeavor to arrange a satisfactory agreement between the railroads and various other interests involved. To this end, Mr. McChesney, president of the Terminal Railroad, appointed Mr. Coffee, engineer Maintenance of Way for the Southern Railway, Mr. Cox, chief engineer of the Wiggins Ferry Company and Mr. Wallace, chief engineer of the Illinois Central Railroad. In conjunction with this commission the writer has gone into the whole matter very thoroughly, and as a result the original plan as outlined is modified as follows:

It is proposed to follow the same plan as first outlined up as far as the new outlet for Cahokia Creek with the same new channel for Cahokia Creek up as far as Horseshoe Lake; but instead of building the levee eastward along the south side of this new Creek channel, to put in a flood gate at its outlet, and build the levee from there northward along the high sand bar on the river front up to the Venice Elevator; thence northward along the high ground to the west end of the embankment of the Merchants Bridge approach; then starting again on the north side of that embankment near the Chicago & Alton Railroad to extend the levee construction northward from the Merchants Bridge to East Alton following the west side of the Chicago & Alton roadbed, unless possibly the Alton & Granite City Electric Line and the Chicago, Peoria & St. Louis Railway should be followed where these lines parallel the Chicago & Alton on the west. At East Alton connection would be made with the elevated track known as the Chicago & Alton cut-off. This would absolutely protect all the area east of this levee, or over 85 per cent. of the total area of the American Bottoms, and practically all railroads, and at a cost, including the new creek outlet, of less than $700 000. The writer has figured that it would be impossible to construct a levee anywhere west of this proposed line of equal height and strength for less than double this amount.

This levee can be constructed and nothing else be done at all, but it would be well that the interior drainage matter should be taken up and carried out at the same time. To this end, the following general plan has been outlined. The Cahokia and Prairie Du Pont districts are to be treated separately, one from the other.

In the Prairie Du Pont it is proposed to connect Prairie

Du Pont Creek directly with the river by a new channel from the village of Prairie Du Pont west, then to widen and deepen the present channel of the creek from there to the foot of the bluffs, all excavated material to be deposited on the north side so as to form a levee from the Conologue Railroad to the bluffs, which will keep this Prairie Du Pont Creek entirely outside of this district as well as the back water from the river. From the eastern end of this new channel a ditch is to be constructed northward up through the lakes and low lands, as far as the Louisville & Nashville Railroad, and suitable connections are to be made with all streams coming out of the bluffs, and proper lateral drains up the various lakes. It was also the writer's recommendation that this main ditch be continued to Spring Lake and that all waters south of the Baltimore & Ohio Railroad be brought south as they originally flowed; but the commissioners of the district have decided to leave the situation as it is north of the Louisville & Nashville and let all these waters go through the flood gates under the Baltimore & Ohio and on into Cahokia Creek. The grade of the bottom of the main channel is to start at the river at elevation 10, or 14 ft. above low water, and proceed on an up-grade of 1 ft. in 6 000 ft., from there on through its entire length, and to be at least 4 ft. below the bottom of the lowest lake in St. Clair County. In the Cahokia district it is proposed to start at the eastern end of the above-mentioned new channel for connecting Horseshoe Lake with the river north of Brooklyn, and to construct a channel up through Horseshoe Lake and along the south side of the Toledo, St. Louis & Western Railway to where that road crosses the creek and then on north by as straight a channel as practicable to the junction of Indian Creek and Cahokia Creek; from there on for two miles north Indian Creek is to be cleaned out and the excavated material used to construct a levee along its west side. At the south end of this district there is also to be constructed a channel from Cantine and Caseyville creeks following the original course of these creeks as much as possible so as to drain all their waters and all streams coming out of the bluffs south of the St. Louis, Troy & Eastern Railroad, directly into Horseshoe Lake. A small levee is then to be built across the south end of Horseshoe Lake, shutting it off entirely from the Cahokia Bottoms in time of heavy rains. The grade of this Cahokia Channel is to start at elevation 10 and have a regular up-grade of 1 ft. in 3 400 ft., about 1.5 ft. to the mile, which will be the same slope that the creek has now, but about 2 ft. less elevation.

In all channels in both of these districts it is proposed that all excavated material be used to construct levees along both sides of the channels so that at a high stage of the river these channels will still carry to the river through the bottoms the run-off from a moderately severe storm without its getting over the levees. In the case of the Cahokia district, however, it is proposed to confine the flood water in Horseshoe Lake and use this as a reservoir in case the river is high and there are heavy rains at the same time. The writer has calculated that a 6-in. rainfall throughout the entire Cahokia district would be held by Horseshoe Lake at an elevation of 35 ft. above low water; and that is the height to which it is proposed to levee across that lake to confine the waters in it if necessary, this being 3 ft. higher than the water was ever known to be in the lake except from back water from the river. In time, of course, this lake will fill up somewhat and at some future day it will pay to levee along both sides of the creek through this lake, but at present it would not. Until this lake fills up more, it is impracticable to use it for any other purpose than above outlined. For this reason the writer does not consider it safe to leave the creek where it is through East St. Louis and put in flood gates at its present outlet; for a stage of 35 ft. on the inside would be a serious matter in the city; hence the change in channel north of Brooklyn. From the river hydrograph for the past thirty-five years it appears that we have never had more than an inch of rain per day with the river stage at or above 30; and all this improvement has been planned to accommodate at least an inch of rain at a stage of 35 ft., something that is as unlikely to occur as a river stage higher than that of 1844. Our rain storms here generally come just before the crest of a flood and not during it. It is estimated that the cost of all this drainage work will be $271 243 for the Cahokia district and $180 000 for the Prairie Du Pont district, or a total of $451 243.

Now, let us compare this with the proposition of bringing the channel along the bluffs. In the first place, as to grade, it would hardly do to raise the grade higher than the present grade of the creek, as laid down by the writer for the channel in Madison County; so let us take the same grade and channel as for the previous plan to start with. If this grade were continued at the same rate, 1.5 ft. to the mile, through St. Clair County, it would strike the river at the outlet at practically the low-water mark of the river at that point, which would be thoroughly impracticable, as it would fill up at once; for, as stated

above, it is found that none of these creeks now maintain themselves at much less than 7 ft. above low water at their outlets, and it is best to get them higher if possible on account of the inefficiency of a deep channel at the outlet at high stage of the river. So, taking about 9 ft. above low water at the outlet, we find that in order to reach the bottom of the present low lands near the south end of Cahokia Creek, in Madison County, we should have to change this grade of 1.5 ft. to the mile to less than a foot to the mile through Clair County, or the same grade as is proposed for the Prairie Du Pont district ditch, but it would be 5 ft. deeper than that ditch, so that through this flattened grade this channel would have to be made wider and higher levees would have to be provided along the sides than in the steeper portion in Madison County.

The cost of this channel would be at least $735 000, or over $280 000 more than the plan previously outlined by the writer. Who should pay this difference? Should the Prairie Du Pont district pay this for the privilege of having the waters from a drainage area of over 400 sq. miles brought down through it, and thus keep its local water from flowing away just so much longer? It would mean that this channel would be full for weeks at a time and thus give little or no chance for the local water to get into the channel. And where would be the benefit to Madison County? North of the Toledo, St. Louis & Western Railroad there is no difference in the two plans; south of that railroad in the first plan we are getting the water away as quickly as possible from the lands that are most damaged by floods, and keeping it away; whereas, in the second plan we are taking all the water directly down through this district that is so hard to deal with, and keeping the channel full of water by reducing the grade and thus not letting the local water get away. In other words, in the channel along the bluffs we are giving less perfect service and at double the cost, for if this channel along the bluffs is ever constructed, Madison County will have to stand the entire additional cost.

As to the Wood River proposition nothing further will be said than that the best grade that can be obtained for bringing it down is less than 0.75 ft. to the mile for about half the distance, and a little less than a foot to the mile for the remainder; the cost not less than $960 000, or more than double the writer's total plan, and with much inferior results. It may be laid down as a general proposition that in nearly all cases it is best to get streams of this kind with steep grades into the main channel

as quickly as possible, and not construct long parallel channels of light grade.

There is one other plan that deserves special mention, and that is the plan of taking Cahokia Creek straight west into the river at about the point where Indian Creek empties into it. Mr. Chas. Sheppard, of Edwardsville, has made complete plans and surveys for this proposition, and during the past few months an endeavor has been made to organize a district and put this through; but so far it has failed and it has been about given up, principally on account of the popular fear among landowners that this might offer an opening for the river to back in and flood the entire country south (of which there is not a particle of danger, however); and then there was no provision made to take care of the local drainage south of this cut-off. This could have been done very reasonably by constructing a small channel from the west end of Horseshoe Lake to the river, along the line suggested by the writer, and then draining Cantine and Caseyville Creeks south through St. Clair County, to which there would probably then be no objection. Mr. Sheppard estimated the cost of this cut-off at $230 000, and the local drainage south could probably be done for $80 000 more, or a total of $310 000, or about $40 000 more than the writer's plan. Either one would be satisfactory, and in fact the writer would rather favor this latter plan if it could be successfully put through.

In these plans for levees and drainage the writer proposes the following details for the work. Levees along railroad embankments shall have a top width of 15 ft. and slope on the outside of 2 to 1 and on the inside of 1.5 to 1; along the river the top width shall be 20 ft., the outside slope 3 to 1 and the inside 2 to 1, and the outside face from top to bottom shall be protected by riprap. All levees shall be constructed of clay except the one along the sand bar from East St. Louis to Venice, where it shall be earth with a facing of clay 5 ft. thick. Some special construction would be required along the river front at the railroad and wagon ferry landings, but this has all been worked out satisfactorily with the engineers of the railroad companies interested. It is intended that the grade of the top of the levee shall be at least 4 ft. above the high water of 1903, which will make it about half a foot above that of 1844. This is undoubtedly ample, but anything less than that is scarcely safe. The bridges for the railroad tracks over the new Cahokia channel north of Brooklyn are to be two span, concrete arches of 30 ft. span each, continuous under all tracks, so as to form a

continuous solid covering. For the drainage matters, it is proposed that the new channel for Prairie Du Pont Creek shall be 30 ft. wide at the bottom with side slopes of 1 to 1; the levee along north side of same 20 ft. wide on top, with side slopes of 2 to 1; the main drainage ditch through St. Clair County 20 ft. wide at the bottom, and the new channel for Cahokia Creek to be 30 ft. wide at the bottom, both to have side slopes of 1 to 1, with levees along both sides 10 ft. wide on top, with side slope of 2 to 1.

For the distribution of the expense of this plan it is suggested that this Drainage District No. 1 of St. Clair County shall construct and pay for all the drainage planned for St. Clair County and the levee up as far as the city limits of East St. Louis, amounting to $180 000 for drainage and $98 500 for levees, or a total of $278 500 for this district, or an average of about $11 per acre. The city of East St. Louis is to build and pay for all levees through the city and up to the Madison County Line, and also for the new creek channel from Horseshoe Lake to the river, as this benefits East St. Louis and no one else. This will amount to $214 000 for the levee and $129 302 for the creek channel, or a total of $343 302, as the total amount to be paid by the city of East St. Louis from the $60 000 per year tax now being levied. North of East St. Louis it is proposed to organize all that country north of Drainage District No. 1 and the city of East St. Louis into a new district, and to build the remainder of the levee and the Cahokia Creek drainage plan, at a cost of $271 243 for the drainage and $257 128 for the levee, or a total of $528 371, or an average of less than $9 per acre. Of course, in this new district, over two thirds of the Chouteau, Venice and Nameoki Levee district will be protected by this high levee, for which it is doubtful if they can be taxed at all unless they voluntarily agree to it in order to push the matter along. This cannot be helped, for this old district has already been taxed for levee purposes and it is doubtful if it could be brought into the new district.

The total cost of this entire levee and drainage plan will be $1 150 000 in round figures. This gives drainage and levee protection for over 85 per cent. of the area of the bottoms, and for a very low cost, easily within the means of the landowners. The levees are on high ground nearly all the way, and for that reason much safer than if built west of this location on much lower ground, to say nothing of the additional expense. The greatest difficulty at present is the organization of this new district, but it is hoped it can all be worked out during the present year.

OBITUARY.

George A. Lederle.

MEMBER OF THE LOUISIANA ENGINEERING SOCIETY.

GEORGE A. LEDERLE was born in Detroit, Mich., September 4, 1858. He studied at the University of Michigan and was graduated in 1881 as a civil engineer.

After graduation he was engaged with George Morrison for several years in bridge construction work, having charge of the erection of many very important structures, including the Union Pacific bridge at Omaha and one at Portland, Ore.

He next entered as partner into the firm of Christie & Lowe, contracting engineers, with whom he was associated since 1892, and personally superintended many of their most important contracts, the latest of which was the construction of the jetties now building at South West Pass, La.

He was a member of the American Society of Civil Engineers, the Western Society of Civil Engineers and of the Louisiana Engineering Society.

John Talcott Norton.

MEMBER OF THE LOUISIANA ENGINEERING SOCIETY.

JOHN TALCOTT NORTON was born October 1, 1864. He early entered the practice of civil engineering.

He was prominently connected with important railway construction in the United States, South America, Mexico and Cuba.

He was chief civil engineer for the Philippine Commission, and while in the Philippine Islands made reconnaissance and report upon important railway work for the Commission.

At the time he became a member of the Louisiana Engineering Society he was engineer for the New Orleans Belt Railroad Commission, which position he gave up to accept the management of an important railroad in Guatemala, Central America, in which work he was engaged at the time of his death.

Mr. Norton was a member of the following organizations: American Society of Civil Engineers, American Railway Engineering and Maintenance of Way Association and Louisiana Engineering Society.

Editors reprinting articles from this Journal are requested to credit the author, the Journal of the Association, and the Society before which such articles were read.

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THE GARVINS FALLS DAM, CANAL AND HYDRO-ELECTRIC PLANT, BOW, N. H.

By George G. Shedd, Member of the Boston Society of Civil Engineers.

[Read before the Society, May 17, 1905.]

Although the power at Garvins Falls, four miles below Concord, N. H., is one of the best on the Merrimack River, it has only recently been fully developed. There has been a dam here since 1815, when a wooden crib dam was built in connection with the Bow Canal, which was the only means of transportation until the advent of the Concord Railroad in 1835. In excavating for the new canal, portions of the old wooden lock-gates were unearthed and found to be quite sound after nearly ninety years' service. A stone dam was built here in 1859, the center portion of which was carried out by the following spring freshet and was not rebuilt until 1879, when, in order to preserve flowage rights, the destroyed portion was replaced with a timber section. This remained until the old dam was destroyed at the completion of the new dam. A pulp mill at the site of the new station utilized a small part of the power here for a number of years. The city of Lawrence, Mass., narrowly escaped being located at Garvins Falls, for its promoters thought very favorably of the situation and power, but could come to no satisfactory terms with the owners of the property or with the town of Bow.

The absolute minimum flow of the river is probably about 1 000 cu. ft. per sec., but only on very rare occasions is the flow less than 1 300 cu. ft. per sec. This gives 4 279 gross h. p. under

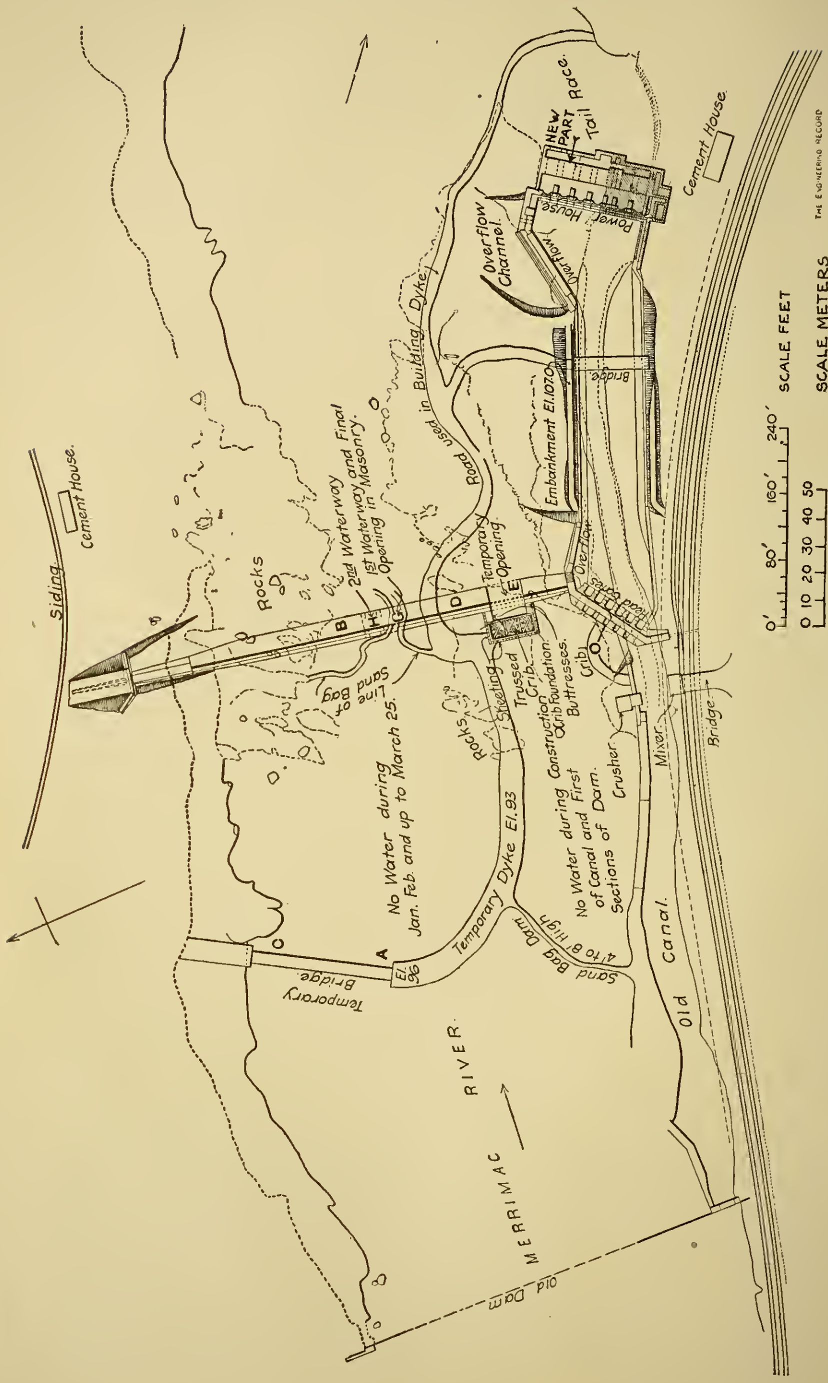


FIG. 1. GENERAL PLAN, SHOWING LOCATION OF CONSTRUCTION PLANT.

a 29-ft. head, or 3 218 h. p. as the ordinary minimum on switch-board, allowing 80 per cent. efficiency of turbines and 94 per cent. efficiency of generators. It is expected from past records that the river will furnish at least 5 000 net h. p. for nine months in the year and for a large part of the time considerably more than this, the average power for the entire year being in excess of 10 000 h. p. It was therefore decided to develop for 6 000 h. p.

As the company's need for power was rather imperative, it was decided to utilize the old dam and canal and build a portion of power station which would be a part of the permanent development. This was completed in June, 1902, and furnished about 1 200 h. p. Early in 1903 it was decided to complete the development. Accordingly, plans were made for new dam, canal, head gates, etc., and for the completion of the power station, and a contract was let on June 5 to the Holbrook, Cabot & Rollins Corporation of Boston.

The work consisted of building an overfall dam of the ogee type with a maximum height of 33 ft., 550 ft. long between abutments and about 800 ft. over all, including head gates; a canal about 500 ft. long and 74 ft. wide at water line, closed at river end by a massive head-gate wall; and making extensive addition to power station.

The location of head gates was practically fixed by the ledge, which furnished good foundation there, but dropped off very materially just above on the line of the old canal. Before adopting final location of dam, several alternate locations, including one curved in plan, were considered in more or less detail. Considering the dam alone, the best location is undoubtedly quite a distance up stream; but when we balance the saving in dam with extra cost of extending the canal, the adopted location is seen to be preferable.

The maximum freshet of which there is any record occurred on March 2, 1896. I have estimated that there was at that time a flow of about 72 000 cu. ft. per sec. This stood according to various authorities between 12.5 and 13.5 ft. on the old dam. It is estimated that the same freshet would cause a depth of about 10 ft. on the new dam. In anticipation of a freshet in the future of even greater magnitude, the dam was designed to withstand safely a flow of 15 ft. over crest. A flow of 100 000 cu. ft. per sec. would cause a probable depth of about 11 ft. 3 in. on crest; and I think it doubtful if that is ever exceeded. No data on coefficient of discharge $C = m\sqrt{2g}$ in formula $Q = m2h\sqrt{2gh}$ for a flow of this depth could be found; but

upon plotting the curve of coefficients as determined by the latest experiments upon crests of this shape, it is seen that they increase very slowly with the high heads. This curve was extended and the coefficient of 4.5 thus determined was used. I am in hopes sometime to have the opportunity to verify this by experiment. The rating of this dam as well as of the wheels has lately been undertaken in coöperation with the United States Geological Survey, so that we expect to have some very accurate records of flow of the Merrimack at this point as well as to obtain data on coefficient of discharge, which seem to be rather scarce, particularly on the high heads.

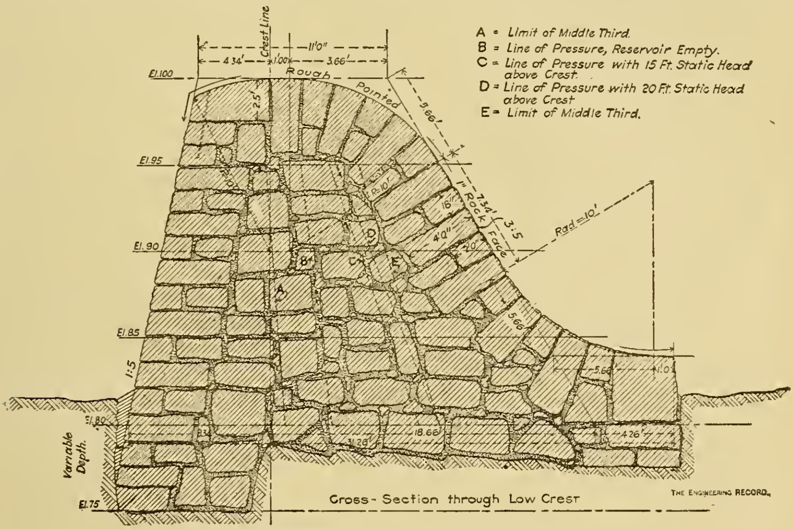
The length of the old dam was 454 ft. between abutments and 473.2 ft. on crest, measuring all offsets of wooden center sections. The length of the new dam was, therefore, fixed at 475 ft. on low crest portion at same elevation (or rather 0.06 lower to allow for slope in river) so as to avoid all legal questions as to flowage rights and at the same time to maintain all the head to which the company is entitled at the low stages of the river. In order to keep the water level down and avoid trouble with railroad, etc., during freshets, an additional length of spillway of 75 ft. at a level 2.0 ft. higher was provided.

As the general level of ledge is about 83 (referring to low crest level as 100) for a considerable distance on both sides of the channel, the toe stone level was made El. 84 except at the ends, where the ledge is higher, and it was stepped up to fit the ledge. For about 75 ft. in the center there is a channel with a depth of 10 or 12 ft. lower than the general level. Several methods of treating the toe at this place were considered, but the one adopted consisted of building an apron of rubble on approximately a slope of 2.25 to 1 from top of toe stone, which was flattened out at the bottom to fit the ledge and surfaced with a heavy split granite paving, the stretchers being at least 2 ft. thick with headers of from 4 to 6 ft. in length.

The hearting of the dam is built of rubble masonry laid in 2.5 to 1 Portland cement mortar. The stones were as large as could be conveniently handled, with a maximum size of 2 cu. yd. or more, and more or less irregular in shape, laid on a thick bed of mortar and far enough apart to insure a good vertical joint. These joints were filled with a rather wet mortar into which were forced as many spalls as could be. Particular care was taken in this respect, because it has been observed in taking down masonry of a similar class that if any poor work occurs it is almost invariably in the vertical joints. The stone used for

hearting was largely a coarse-grained granite obtained from the canal and dam excavation, although some mica schist from excavation was used which was found to be heavier than granite.

The upstream facing is of granite rubble well bonded to the hearting. The downstream facing is of heavy ashlar. The stretchers have a minimum depth of 2 ft., a minimum length of 3 ft. and a rise of from 18 in. to 24 in. The headers, which were placed alternately between every two stretchers in each course, were at least 4 ft. long with a minimum face length of 2 ft. The total face area of headers is about one sixth face area of each course. Capstones are 2.5 ft. deep at crest and at least 4 ft.



LOW-CREST PORTION OF MAIN DAM.

FIG. 2.

long. Toe stones are very heavy, being at least 4 ft. long, 4 ft. deep and 3.5 ft. on face.

The foundation ledge was so seamy that the original plans of cut-off trench could not be strictly followed; it was usually somewhat wider, with the downstream face taken out on more or less of a slope depending on the seams encountered. The foundations were everywhere carried down to good solid ledge and all loose rock was removed by barring and wedging. Before laying masonry the ledge was thoroughly washed and all seams were grouted.

Construction of the canal was well along before the dam was started, but perhaps it is better to describe each separately, regardless of sequence of construction.

Work on the dam was commenced on the foundation at both ends in August, and the first stone was laid on September 8, 1903, near north abutment. The ledge was very seamy and required great care in blasting. There is a small low island just above the dam site which is entirely submerged at high water, but at ordinary water divides the stream into two parts, by far the larger portion following the channel in the center of the river. Advantage was taken of this by the contractors, who first closed the channel on the south side of the river at the head of the small island referred to, by building a sand-bag cofferdam from 4 to 8 ft. high and 200 ft. long, and also building a low cofferdam along the center of the island. This permitted them to build the head gates, south abutment and about 145 ft. of the dam foundation comparatively free from water. Two sections of about 50 ft. in length at the south end of the dam were built to full height, and a gap left between of about 60 ft., with foundations in to about the level of the ledge. This gap was opposite the channel on the south side before referred to, and was designed to take at low-water periods all the water that could not be sent through the canal.

After these sections were in and the headgates and south abutment above water, the sand-bag dam was removed and a cofferdam built from the head of the island across the main channel to the north shore about 400 ft. above the dam. This would have been quite a proposition at any time of the year, but it was especially difficult in the cold weather of December and January, anchor ice and a foundation of bowlders making it hard to get a tight job. It was satisfactorily accomplished, however, early in January, and the water diverted to the south channel.

In the meantime work on the dam had been going on at the north end, where it was now nearly of full height for a distance of 160 ft. from the north abutment. This left a center section of 245 ft. to be built.

As the company was anxious to use power at the earliest possible time and the spring freshets were almost certain to destroy the cofferdams, it was decided to push work through the winter while the water was low, and to attempt to raise the center section to such a point before freshets came that in case the cofferdams were carried out, the dam would be high enough of itself to divert the low flow through the gap at the south end previously mentioned, and that the work could proceed after the freshets had subsided without rebuilding coffers. If this

could be accomplished, at least six months could be saved. Accordingly, every energy was bent on getting the foundation prepared, which proved to be no easy task under the almost arctic conditions then prevailing. As soon as a section of foundation was prepared, all the masons that could be conveniently worked started on the masonry. The foundation was put in for the whole width of the section up to the level of bottom of toe stones, and then the upstream masonry approximating about one-half the thickness of the dam was carried up to El. 92, or 8 ft. below crest. At this elevation it was expected that there would be sufficient waterway together with the canal and 60 ft. gap to handle an ordinary spring freshet without danger. The flow was expected to be deep enough to pass ice over unfinished masonry without damage.

The leakage from the main cofferdam was collected by several lines of sand bags and carried across the foundation in a channel prepared for it over the ledge (at G, Plate I), allowing the foundation to be put in on both sides. The flow was then shifted to a point (H) over the masonry and finally back to the first location, where in the meantime the foundation had been completed and a gap 8 ft. wide left in the masonry above the foundation, which was not closed until just previous to effecting the final closure at E. Provision for closing this 8 ft. gap was made by building a 12 by 12 hard pine timber frame into buttresses built out about 3 ft. from the face of the dam, against which was later placed 4-in. tongued and grooved sheeting driven into sand bags at the bottom.

By the 10th of March the masonry was up to El. 93 or higher everywhere, except at the two gaps noted; and all derricks and machinery were moved from the river bed in preparation for the freshet which was expected at any time, but did not come until the 28th of March; it caused a flow of 9 ft. over the crest of the unfinished center portion, which stood the test without the slightest damage. Of this masonry the last portion had been finished only eighteen days and no portion over two months, during which time the temperature ranged down to 24 degrees below zero, which would seem to prove that Portland cement masonry construction can be successfully carried on in freezing weather if proper precautions are observed.

A series of experiments on the effect of freezing mortar was made during the progress of the work. The mortar used was of 1 part Lehigh Portland cement to 2.5 parts sand, and was taken from that being used at the dam. The mortar for each experi-

ment was made into fourteen briquettes, half of which were placed in water in the office and the rest placed out of doors in the air. The sand and water were both heated, and salt was used in the water in the proportion of 4 lb. to a bbl. of cement. The average temperature during the period of these experiments was considerably below freezing, and in some of the first experiments the briquettes were frozen continuously for over three months, but seemed to gain in strength at about the same rate as those that were thawed out earlier. In one case a temperature of 24 degrees below zero fahr. occurred within 12 hr. after the briquettes were made, but the final strength was not affected. The results of experiments made between November 28, 1903, and February 27, 1904, are as follows:

TENSILE STRENGTH.

7 Days.		28 Days.		3 Months.		6 Months.	
Water.	Air.	Water.	Air.	Water.	Air.	Water.	Air.
In lab.	Outdoors.	In lab.	Outdoors.	In lab.	Outdoors.	In lab.	Outdoors.
203	90	297	173	314	320	374	493

It will be seen that at periods up to about three months the strength of the frozen mortar is materially reduced, but at three months it is fully equal to mortar that has not been frozen, while at six months it appears to be even stronger. This same increase was observed by Mr. T. F. Richardson in an elaborate series of experiments made at the Wachusett dam, an account of which he has recently published.

Masonry work was carried on at the dam when the temperature was as low as 5 degrees fahr., but the following precautions were observed whenever it was below freezing: the sand and water were heated, salt was used in the water to the extent of 4 lb. per bbl. of cement, all large stones and spalls were thoroughly steamed before laying. Great care was taken to have the surface of old work thoroughly cleaned of all ice and dirt. At night all fresh work was covered with large tarpaulins and a steam jet kept under them.

Work was finally resumed upon the center section of the dam on May 25, 1904, after several unsuccessful attempts on account of high water. At this time there was about a foot of water flowing over the uncompleted portion. A sand-bag coffer

was placed on the top of the dam and at the upstream edge of the masonry and work carried on behind it until the water receded enough to all pass through channels prepared for it. It did not take long to complete the center section in this manner, close the 8-ft. gap previously spoken of and build the apron at the toe, after which it became necessary to close the 60-ft. gap at the south end.

The contractors had here previously built masonry buttresses on each side of the opening, extending about 15 ft. upstream, and also prepared a foundation for the closure crib. The lower trusses of this crib were floated into position successfully on July 15, and it was gradually built up above water and loaded with stone, after which the sheeting was driven, allowing water to rise and flow over the new dam.

The masonry in the final closure was completed on August 14, about eleven months after the first stone was laid and one year after work was started. After the dam was completed, the old dam was destroyed by dynamite.

The new canal, which was started June 8, 1903, was practically an enlargement of the old canal; it is about 500 ft. long and 74 ft. wide at the water line, with a depth of 12 ft. below the crest of the dam and a total depth of 17 ft. at head of canal, 18 ft. at head of fore bay and 22 ft. in fore bay. It was designed to furnish 2 900 cu. ft. per sec. with a mean velocity of 3.55 ft. per sec. when water was at the crest of the dam. Under these conditions the loss of head in the canal was calculated to be .07 ft. due to velocity. The loss in head gate opening was estimated at .25 ft., making a total loss of head from pond to rack of .33 ft. The excavation of the canal was largely in rock. Above the ledge and in a number of places where the ledge was not suitable, the canal was lined with a dry rubble wall which was 3 ft. thick at the top with a face batter of 6 in. per ft. and a rear batter of 3 in. per ft. This was made of large, smooth stones with close joints to reduce friction as much as possible. At the upper end of the canal near the railroad there is a curved retaining wall built of concrete. At the opposite side is an overflow 45 ft. long with its crest 3 ft. above the crest of the dam. At the lower end the canal widens out into a fore bay 136 ft. wide, with an overflow 90 ft. long with crest 2 ft. above the dam. Both overflows are provided with flashboards to a height of 5 ft. above the crest of the dam and bridges over them from which flashboards may be operated. The flashboard standards are removable to aid in floating out large cakes of ice. One bad

feature of a flat top overflow was demonstrated this spring when the station men were attempting to float out ice about 30 in. thick. A large cake grounded on the outer edge and tilting up broke the bridge stringer. If the top had been sloped off this probably would not have happened.

A waste gate 10 by 12 is provided just above the rack, which is used principally to float out anything lodging against the rack. Means for drawing off the canal is provided by a 4 ft. by 6 ft. gate with sill at bottom of fore bay. The rigging for these gates is operated by hand. A massive head-gate wall is built at the head of the canal with six arched openings 10 ft. wide and 14 ft. high with 5 ft. piers between, rounded at the upper end to make easy entrance for water. The sills of these gates are on a level with the canal floor, or 12 ft. below crest of dam. The top of the wall is 8 ft. wide and 17.5 ft. above crest and about 38 ft. above lowest foundation. The piers have a total length of 30 ft. at the base, 8 ft. of which are above the upstream face of the wall and contain the grooves for gates and stop-planks or screens. The lower part of the head-gate wall up to 3 ft. above the crown of the arches is of $1-2\frac{1}{2}-4\frac{1}{2}$ Portland cement concrete; above that point is of rubble concrete. The openings are closed by 12 ft. by 14.5 ft. hard pine gates armored with angle irons on the bottom and on the face next to the guides. The guides are of cast iron built into concrete. A cut-stone sill is set under each gate. It is not expected that these gates will be absolutely tight, but it is not necessary that they should be. The rigging to operate gates is unusually heavy, for it was considered best not to depend on any of the numerous roller-bearing schemes to reduce friction, which are all more or less liable to be affected by ice, but to put in simple, heavy gates and provide power enough to operate them under all conditions.

A line shaft extends the whole length of the gates, which is operated by 2-10 pole, 7.5 h. p., 440 volt motors running at a speed of 720 rev. per min. Either one or both motors can be run, and one or all six gates thrown in by means of friction clutches. It takes 20 min. to open or close gate the full height of 14 ft.

As it was essential to have the old station shut down as short a time as possible, the canal work was rushed to the greatest possible extent. The earth was first excavated to water level by means of teams, then both banks were lined with derricks so that work was progressing the whole length of the canal at once.

Fig. 3. Dam during Construction, June, 1904.

Fig. 4. Dam and Head Gates, August, 1904.

Most of the earth and rock not suitable for masonry was loaded directly on to cars and used in raising the Boston & Maine Railroad tracks, which we were required to raise for a distance of about a mile and one-half to a maximum height of about 2 ft. This was necessary because the tracks opposite the new dam were 2 ft. lower than at the old dam. The railroad company afterwards raised these tracks an additional foot here.

The old station was shut down on the 6th of July, 1903, and by the 19th of November, everything being in readiness, the water was let into the new canal for the first time and the wheels started again.

As work was in progress upon the dam, it was necessary, in order to utilize the station, to provide a temporary connection from old canal to new canal. This was done by building a curved crib cofferdam made of railroad ties driftbolted together and loaded with stone. This was faced with 2-in. plank and battened to make it tight, the lower end of the plank being set in concrete. The cofferdam was later removed when under water by grapple. After the closure cofferdam in the 60-ft. gap in the dam was in place, the remaining three head gates were raised and water taken directly from the new pond. The new wheels and generators were then started, and together with the old wheels and the waste and draw off gates wide open were able to keep the water from flowing over the dam for the greater part of the day, which greatly aided work on the closure.

The station is a brick building on a granite foundation, containing seven arched openings, six of them about 15 ft. wide, to allow tailwater to pass freely from draft tubes to tailrace. The old part is about 65.5 ft. by 67 ft., and the new part 84 ft. by 67 ft. The old part was built in 1901 by Head & Dowst of Manchester, N. H. The new station is of similar construction, new part is carried down to grade 64 on account of seams in the except that the foundation for the fore-bay building wall in the ledge. In the old part they were only carried down to grade 80 and spaces for draft tubes channeled out in ledge. The top of this wall is at grade 108.5, and it contains six openings 12 ft. in diameter, into which are built the wheel cases. Each pier between the openings is reinforced by a buttress.

The generator room is 30 ft. by 146 ft. wide, with floor level at grade 35. As the backwater has been known to be somewhat over 5 ft. above this elevation, it was waterproofed to a point 7 ft. above the floor. The wheel room between the fore-bay wall and generator room is 33.5 ft. wide, in which the

water may rise at periods of unusually high water. The shafts from the wheels are located in this room, passing through stuffing boxes in water-tight bulkheads in the wall of the generator room. The walls of the room are 24 in. thick up to the level of the track and are 8 in. above this. The room is covered with a pitched slate roof. The wheel room roof is nearly flat at a lower elevation.

The station is designed to contain 6 units rated at 650 kw. each, which Mr. Richardson will describe in detail. Four of these are at present installed and the other two will probably be put in in the near future. A steel rack 20 ft. high with 3.5 in. by 0.25-in bars spaced 1.25 in. apart is placed just above fore-bay wall to protect the wheels. This is designed strong enough to act as a dam in case it is clogged by anchor ice and the water drawn away from behind by wheels. A short tailrace is provided from station to river in which the loss of head is about 0.08 ft. In constructing this the greater part was done by team work and the balance by orange peel dredge.

Hollis French and Allen Hubbard were the consulting engineers for the work, and Holbrook, Cabot & Rollins were the general contractors. Too much credit cannot be given them for the able and satisfactory manner in which they carried out the work.

The new station above the foundation was sublet to Maguire & Penniman, of Providence, and the steelwork to the Boston Bridge Works. The plant used consisted of fourteen large guy derricks each fitted with swinging gear and double drum Lambert or Lidgerwood hoisting engines, seventeen Ingersoll-Sergeant steam drills, one Farrell stone crusher, one Carlin's 5 ft. cubical concrete mixer, one 10-in. Lawrence centrifugal pump, one 5-in. Cameron pump and several 4-in. pulsometers. During the busy period there was an average force of about 300 men employed.

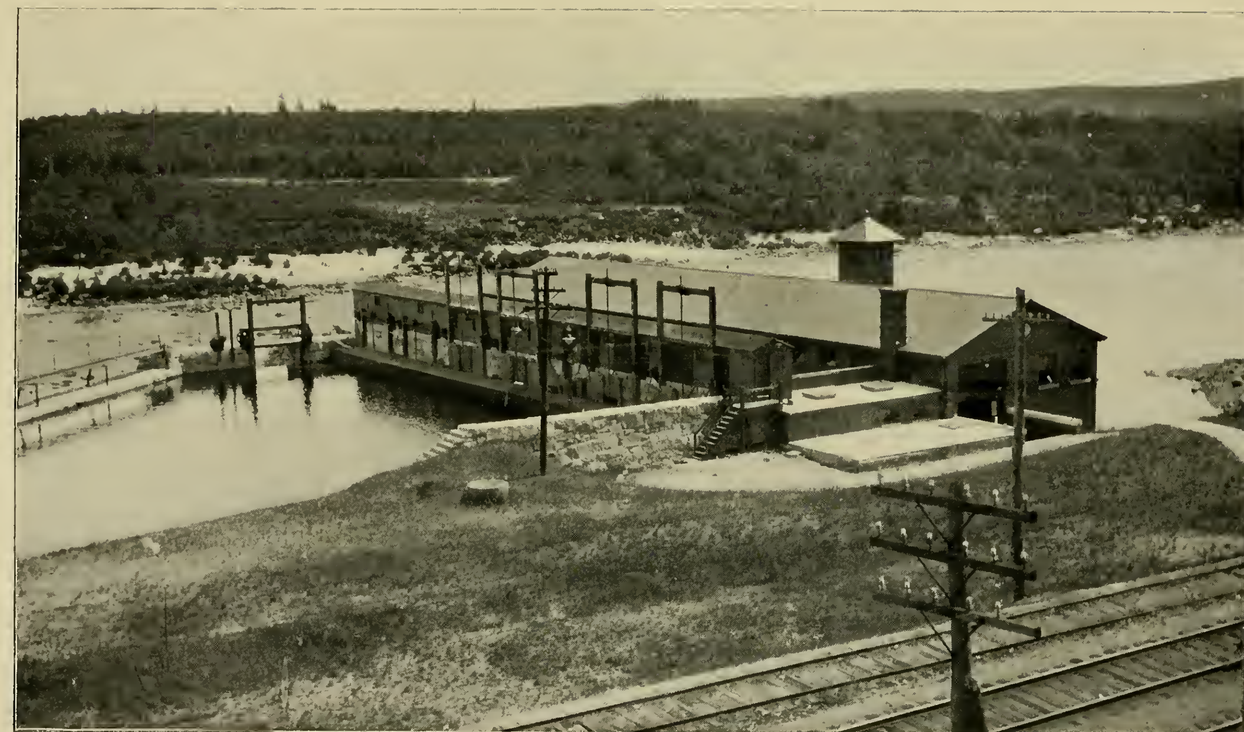


Fig. 5. Completed Station, 1905.

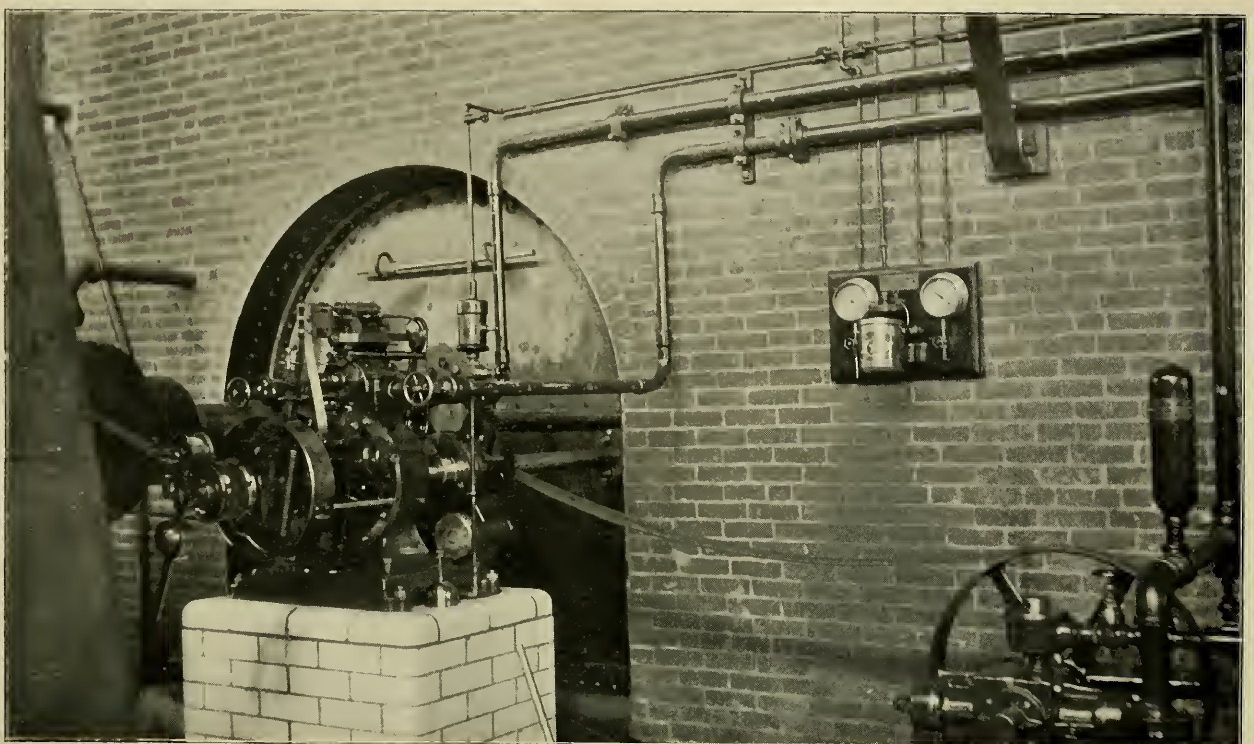


Fig. 6. Sturgess Governors.

HYDRO-ELECTRIC DEVELOPMENT AT GARVINS FALLS, BOW, N. H.

By Edward B. Richardson.

[Read before the Boston Society of Civil Engineers, May 17, 1905.]

General Scheme and Conditions.

In considering the plan of the Garvins Falls station it should be borne in mind that it was built in two parts, the original and the addition. This fact accounts for the unsymmetrical layout, the exciter units, the switchboard, toilet rooms, etc., not being at the center of the station.

The total capacity of the plant in generating units will be 3 950 kw. when all the machinery planned for is installed, this being all the Merrimac River is good for commercially at this point. The aggregate horse power of the turbines will be 6 565 at 26 ft. head and more at 28 ft.

There are 5 393 sq. ft. of generator, exciter, governor and switchboard building area, and 4 506 sq. ft. of turbine building area. These areas are obtained from outside building dimensions. The above, together with certain additional space (furnace room and coal bin) not included, gives a total of 10 683 sq. ft. ground area and, considering the plant as it will be at normal rating, gives 2.709 ft. per kw. capacity.

The units are horizontal direct-connected ones of 650 kw. capacity, and the current is generated direct at the high tension of 12 000 volts, thus obviating the use of step-up transformers.

The power developed is transmitted to Manchester, about 14 miles away, where it enters a sub-station, passes through protective apparatus to step-down transformers and goes from them at lower voltage to a distributing switchboard. From this board it is sent out after any further transformations necessary for various uses, among which may be mentioned incandescent lighting, arc lighting, street railway power and mill and factory motor drives.

Into the same sub-station and to the same switchboard power is delivered from other plants of the company, there being three water-power plants and one of steam power, the latter being held for reserve purposes at times of breakdown or low water. From this it will be seen that the Garvins Falls plant is only part of a large system.

Turbines.

There are 4 triplex turbines of over 1 000 h.p. and 1 small duplex one of 75 h.p.; later two more large units will be installed, provision having been made for them.

All turbines are Rodney Hunt Machine Company's make, McCormick design of runners or wheels.

Each large unit has three 39-in. runners, mounted in the horizontal shaft which revolves at 180 rev. per min. Two of the wheels in each set discharge through a common T center

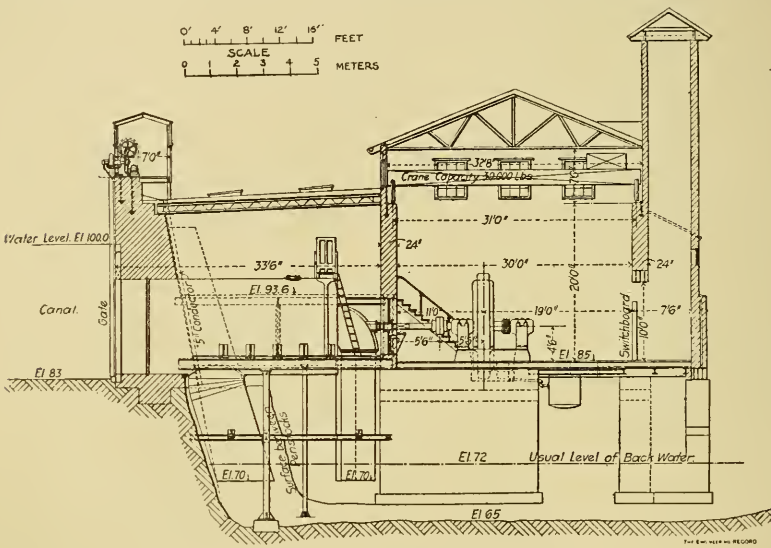


Fig. 7. Section of Power House.

and draft tube near the fore-bay wall; the third wheel is set opposite the quarter turn at the downstream end of casing, through which it discharges into a smaller draft tube.

The draft tubes shown in the section are of equal diameter throughout their length, but those for the later units increase in diameter 12 in. in their length, about 15 ft., allowing a freer discharge. In all cases the ends are horizontal and extend about 2 ft. below the level of usual tailwater. They are hung from the casing and supported and braced by I-beams.

The wheels, cylindrical gates, bearings, etc., are inclosed in a 12-ft. diameter, $\frac{5}{16}$-in. iron plate penstock or casing provided with a heavy cast-iron end, the latter in two concentric parts, the outer fixed, the inner removable. To this last the quarter turn previously mentioned is tap bolted. The entire outfit

rests on two I-beam girders, one on either side of the casing, which are supported at each end on masonry and in the center by columns.

In the later design the T-center and certain other weight-bearing parts are stiff castings which rest directly on the girders and to which the outer casing is bolted. The earlier design provided plate T-centers and less heavy bearing parts which were riveted to the casing, the weight all being taken on brackets riveted to the latter and resting on the girders. The newer design gives much more rigid construction and prevents any shifting of bearings from change in shape of casing due to additional weight of water.

The penstocks are provided with drain plugs for entirely evacuating water after the shut-off gates are closed. Each also has a large combined manhole and suction valve and a relief valve, the latter being piped to a point below the floor. As far as known the relief valves have never operated, the suction valves probably have opened to a small extent.

The gates for the runners are plain cylindrical ones without fingers, and are operated together for each unit by the governors through a shaft, bevel gears, racks and pinions.

For supporting each main shaft there are four bearings in the penstock and quarter turn, two of the bearings taking the end thrust due to tendency of the unbalanced third wheel to work along with the water. The wear due to this thrust is not a serious matter and varies with the amount of sand in the water according to the condition of the river. All bearings are of green oak on end and are lubricated by the surrounding water only.

Between the turbine and generator rooms there is a heavy cast-iron bulkhead for each unit. This bulkhead is in the shape of an inverted U, the center part being split horizontally and tap bolted to the frame, the joint being machined. It carries a stuffing box for the gate shaft as well as one for the wheel shaft, thus making a watertight partition between the two rooms. This is necessary as high back water will rise above the turbine room floor, no attempt having been made to keep same watertight. On the generator room side of the bulkhead is a heavy ball and socket oil ring bearing for the main shaft; beyond this bearing the shaft is extended to the face of the generator coupling.

It should be stated that the removable part of the cast-iron end of the penstock and similar part of the bulkhead

afford an opening large enough for the removal to the generator room of all running parts and most of the other parts of the turbines.

The shafts in the earlier units were in one piece, those in the later ones being in two pieces for ease of handling.

The top of the penstock opening is 4.5 ft. below elevation 100, the top of the dam, near which elevation the canal water stands, but no trouble has been experienced with sucking in of air.

The guarantees on the turbines were as follows:

Original Units.

Generator turbines, 1 047 h.p., at 26-ft. head.
Executor turbine, 75 h.p., at 26-ft. head.

Additional Units.

Generator turbines:
1 035 h.p., at 25-ft. head.
1 099 h.p., at 26-ft. head.
1 162 h.p., at 27-ft. head.
1 228 h.p., at 28-ft. head.

Penstock Gates.

For each large penstock there is a wooden gate of selected Georgia hard pine, 8 in. thick. The stems are at the ends of the gate timbers and are bolted to same.

The timbers are fitted closely together and have a 1-in. by 2-in. soft pine spline. There are two vertical draw bolts extending through all the timbers with eye heads at upper ends.

Each gate is 14 ft. 4 in. wide by 12 ft. 9 in. high over all. Wearing surfaces are faced with $\frac{5}{16}$-in. flat iron screwed to the timbers. This reduces the friction considerably and does not increase the leakage perceptibly.

These gates run on wooden frames, similarly faced with iron, which are built and anchored into the wall, the faces projecting about 1 in. beyond the line of the masonry.

Each gate is provided with an 18-in. by 36-in. filler or wicket gate which is operated by a hand wheel on the threaded end of a shaft, the weight of the shaft and gate being taken by ball bearings.

The object of a large filler gate is to allow enough opening for water to fill the penstock rapidly and to keep it full even should there be considerable leakage through a broken wheel gate. By keeping the penstock full of water the pressure on the gate is, of course, reduced and allows easier handling. The advisability of this has been demonstrated in this plant.

Usually the gates are operated by a 5 h.p. General Electric Company 3-phase induction motor which is back geared to a shaft running the length of the penthouse. By means of friction clutches and link belts connection is made to the hand operating shaft on the gate mechanism. It was found by experience that the worms on this gear should be of steel to obtain requisite strength.

Generators and Exciters.

The station is now equipped with four 650 kw. General Electric Company non-compounded, type A.T.B., 40-pole, 60-cycle, 3-phase generators, revolving at 180 rev. per min., provision being made for two more units as stated.

The generators are capable of standing an 8-hour, 25 per cent. overload without undue heating and a larger overload for shorter periods.

The stator dips down below the floor level so that a pit was necessary, and into this pit, from conduit laid below the floor surface, are brought the wires from the switchboard.

The generators may run in parallel on either set of bus bars or they may be separated, part on one set, part on the other set of bars. They operate very satisfactorily with one another and also with those in the other plants of different design.

For the excitation of the generators is provided a 50 kw. 125-volt direct current General Electric Company generator direct connected to a water wheel. Another similar exciter is also provided, being mounted on a common bed plate with and direct connected to a 75 h.p., 3-phase, 440-volt induction motor of same make. Each exciter is capable of furnishing sufficient excitation current for 4 large units, and as provision is made in the station design for another motor-driven exciter there should be no occasion for a shut down from exciter troubles.

The exciters are similar electrically as well as mechanically, so that parts are interchangeable. This makes it possible in case of needed repairs to substitute parts without the delay of sending to the factory.

By using General Electric Company type T.A. regulators on the exciter bus bars the regulation of the plant has been improved to a marked degree, so that it would seem advisable to make provision for some such device in similar plants in addition to the best mechanical governors.

The Governors.

In the original plant on the large units are provided type B Lombard governors; on the exciter unit type D is used.

The difference between the two types is only in the power developed, the latter unit requiring fewer ft-lb. to move the gates.

Each governor is direct connected to its horizontal gate shaft, extending through the bulkhead, by means of a pin clutch.

Speed and power connections are made through belts from the wheel shafts. The governors are set up on enamel brick foundations to raise them to necessary height for direct connection, thus obviating gearing.

The governors give very good satisfaction, but like all mechanical contrivances have their faults. As stated, the wheel exciter has a similar governor operating in like manner. The only difference in plan being that on account of the high speed, 600 rev. per min., of the wheel shaft, the pump or power drive is belted through a counter shaft to obtain necessary reduction. This latter governor is not ordinarily used, as an electric regulating device, previously mentioned and again spoken of, takes care of any reasonable load changes, greater changes being made by hand control of the gates. The above governors do not show in any illustration. All gate shafts in the entire plant can be operated by hand if necessary.

Considering the newer units a change was made in make of governors. Here type O 30-in. by 26-in. governors of the Sturgess Governor Engineering Company are used. They are smaller and the company favors them, although at present they are not satisfactory on account of oil leakage. This trouble will soon be remedied by new valves and seats of better design.

These units are not self contained, a pump for each governor being mounted on a separate foundation. The two pumps for a pair of governors are located close together and connect by piping to a common tank mounted on the wall. This tank is large and has capacity for three units. On the installation of the future machines it is planned to duplicate the present Sturgess outfit, cross connecting the tanks and thus affording reserve power should a pump break down.

These latter governors have a remote control device operated from a switch on the proper switchboard panel. On the board is a double pole, double throw switch which remains open

unless held closed. By throwing the switch up an electric connection is made to the $\frac{1}{10}$-h.p. motor mounted on the governor, causing it to open the wheel gates by operating through gears on the valve stem. By throwing the switch down the gates will be closed. The principle of these governors is similar to that of the Lombard in that oil pressure admitted to the cylinder by means of a valve controlled from the governor operates the piston. The cause for change of type of governors was chiefly one of price, although the desire to obtain a more perfect governor was held in mind.

Switchboard.

This is comprised of blue Vermont marble, 2 in. thick, set in a built-up oak frame, the wood being thoroughly treated with a fireproofing solution and then well shellacked. The advantage of a wood frame over the usual construction of iron is in its non-conducting qualities.

Over all, the board is 43 ft. 6 in. long and is divided into 15 panels. It stands opposite a bay and is 3 ft. out from the building wall and at a distance of 10 ft. 6 in. from the bay wall.

A complete panel is provided for each generator, exciter, feeder, for motors and transformers and for station and outside lighting.

On the various panels are mounted instruments for measuring voltage, current, energy, power factor, frequency, etc., and switches for controlling the different apparatus. Each high-tension connection to the bus bars is made by an automatic overload switch.

All wiring is done with fireproof covered wire, only secondary or low-tension wires being on the board. All high-tension wiring, transformers, switches, etc., are on the rear wall, the oil switches being separated by alberene stone slabs through which the two sets of high-tension bus bars pass, the wires of each set being in a vertical plane. Current and potential transformers are located above or below the latter switches as most convenient for the wiring.

These switches were made by the Condit Electric Manufacturing Company and this company supplied the board complete. Connections between the operating handles in the front board and the high-tension oil switches proper on the rear wall are made by bell cranks and rods, the latter passing through

slots in the floor. Between these slots the floor is built up of vitrified conduit laid in cement, through which the wires pass.

The generator rheostats are mounted above the board on the wall, mechanical operating connections being made by sprockets and bicycle chains. As stated, any generator may be thrown on to either set of bus bars through one of its triple-pole, single-thrown oil switches. Certain panels are left blank for future use.

All the secondary circuits, these being the ones on the switchboard, are grounded so that chances of dangerous shocks are reduced. On each end of the board as a wing panel are mounted the type T.A. regulators previously mentioned, for controlling the exciters as the generator loads vary. These regulators give fine voltage control to the plant and aid materially in operating the entire system.

On the rear wall, above the oil switches, but below the slate panels set in the building wall, through which the line wires enter, are mounted the lightning arresters, a set for each line.

Ground connection is obtained by a copper plate immersed in the tailrace water and also by connection to the wheel casings.

General Equipment.

The station is equipped with a 15-ton hand-power crane, having a span of 31 ft. center to center of rails, built by the Whiting Foundry Equipment Company. The crane was found exceedingly useful in installing the additional machinery, and in case of large repairs and new installation is practically indispensable.

Primarily for taking care of leakage into the station there is a $2\frac{1}{2}$-in. centrifugal pump mounted on an enamel brick foundation in the generator room. The pump is direct connected to a 2 h.p., 3-phase, 440-volt induction motor, revolving at 600 rev. per min. As a secondary consideration the discharge of the pump has fire-hose outlets, with hose attached. Should no power be on the plant when pumping is necessary an auxiliary hand pump may be used.

The transformers for reducing the voltage from 12 000 to 440 volts for motor uses are oil cooled, type H, each of 25 kw. capacity of General Electric Company make and are located under the entrance stair platform; three such transformers, any two of which are large enough to furnish the power necessary for the exciter motors, are ordinarily connected.

In the switchboard bay is located a single transformer for station and other lighting. The lighting switches on the board are so arranged that should this transformer burn out, direct current from the exciter bus bars can be used.

Station lighting is by a row of incandescent lamps along both walls, about three fourths of the way up to the crane tracks. The power used for the head gates, penstock gates and pump are all supplied from the three transformers used for feeding the exciter motor as mentioned above.

Editors reprinting articles from this JOURNAL are requested to credit the author, the JOURNAL OF THE ASSOCIATION, and the Society before which such articles were read.

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ANNUAL ADDRESS.

BY GEORGE E. MOULTHROP, PRESIDENT OF THE MONTANA SOCIETY OF ENGINEERS.

[Read before the Society at Butte, Mont., January 14, 1905.]

Members of the Montana Society of Engineers: Following the mandate of your constitution and the precedent of some of my predecessors, I have attempted to collect for this meeting some data covering the engineering work and progress in our state during the past year. That which I have to present to you is but a small part of the whole, for I find from my inquiries that going on in all the industrial plants, quietly and unostentatiously, are engineering inventions and improvements gigantic in their material results, so varied and numerous that each one, in an intelligent, detailed description, would occupy by itself more time than is allotted at a meeting of this kind for more general passing reference — touching lightly upon the many.

So it happens that, in this compilation, the inquiries that were made, having developed so many and unexpected interesting engineering items, one is hopelessly at a loss as to how much of this or how little of that could be fittingly wedged in; and manifestly must content himself with only a modest part of what the mining, metallurgical, electrical and mechanical engineers have accomplished in the state since your last annual meeting. I realize, too, that that which is not mentioned may, perhaps, be more interesting and important than the pages which fill the allotted time, and hope that those who are successfully solving these engineering problems and who could present them

in an accurate and attractive paper, may be induced so to do for your regular meetings during the year, giving to their engineering brethren the benefits of the knowledge acquired by their experience and constant study to create new, and improve old, methods.

Railroad Work.

Mr. Wm. Ashton, chief engineer of the Oregon Short Line Railroad Company, says that his company has done little work during 1904 other than ordinary maintenance. Twenty and three-tenths miles of new 80-lb. steel rail were laid from Dillon north, a new roundhouse and shop plant, costing about $80 000, were completed at Lima and several small wooden bridges were replaced by cast-iron pipe with concrete end walls.

The Great Northern Railway Company completed its Columbia Falls — Rexford — Line. The construction of this line shows the modern day railroad tendency to decrease curvature and grades even at the expense of a longer line. The new line starts at Columbia Falls and runs northwesterly to a junction with the Montana & Great Northern Railroad at Rexford on the Kootenai River. Thence main-line traffic will take the Montana & Great Northern Railroad, south, along the Kootenai River to Jennings, a station on the present main line of the Great Northern.

The following table gives the important data of the old and new lines:

	Old Line.	New Line.	Difference.
Maximum degree of curvature,	10 deg.	3 deg. 30 min.	6 deg. 30 min.
Total curvature,	9 646 deg.	4 216 deg.	5 430 deg.
Maximum grade, west bound,	1.5 per ct.	0.25 per ct.	1.25 per ct.
Maximum grade, east bound,	1.5 per ct.	0.7 per ct.	0.8 per ct.
Total rise and fall,	3 790 ft.	1 490 ft.	2 300 ft.
Length of line,	95.2 miles	111.9 miles	16.7 miles
Haskell Pass tunnel elevation,	4 148 ft.		
Dickey Lake, summit elevation,		3 285 ft.	863 ft.

The differences all being in favor of the new line, except the length.

The line between Columbia Falls and Rexford involved the removal of about 5 000 000 cu. yd. of material, the construction of 22 small bridges and 2 short tunnels. The line is laid throughout with 77½-lb. rails. The territory traversed by the new line contains no towns. A division point is established at Whitefish, 8 miles west of Columbia Falls. Terminal facil-

ities besides yard, station, coal chute, roundhouse, etc., include a combination two-story warehouse and office building. This building, the roundhouse and oil house, are built of concrete blocks. There are 14 towns on the main line, of which Kalispell is the largest. This section of the old line from Columbia Falls to Kalispell will form part of a branch extending southward to Somers at the head of Flathead Lake, where large sawmills and a timber-preserving plant are located. Probably this branch will eventually be extended southerly along the east side of Flathead Lake to connect with the Northern Pacific at Jocko. The remainder of the old line from Kalispell to Jennings will probably remain in service for some time, at least until the timber tributary to it has been removed.

We are indebted to Mr. A. H. Hogeland, chief engineer of the Great Northern Railroad Company, and to the *Railroad Gazette* of September 9, 1904, for the above data.

Mr. F. J. Taylor, Northern Pacific division engineer at Livingston, Mont., writes us as follows regarding the work of his company: "Work has consisted mostly of betterments of the line, of which the following are the principal items: Five steel bridges have been built to replace timber structures, two crossing Glendive Creek near the town of Glendive; one crossing the Yellowstone River on the Rocky Fork branch, and crossings of the Madison and Jefferson rivers on the Butte line. In connection with building the last two-mentioned bridges, changes of line were made for improving the alignment.

"About 50 smaller timber bridges have been replaced by concrete culverts and other permanent construction.

"The work of filling trestles on the mountain on the Butte line, which has been in progress during the past three years, was practically completed during the past season. This disposed of 18 000 lin. ft. of timber trestles and required the handling of 1 600 000 cu. yd. of material.

"The O'Keefe and Marent viaducts, west of Missoula, have been reinforced to make provision for the heavier loading for which bridges are at present designed; O'Keefe viaduct being 925 ft. long and 112 ft. high and Marent viaduct 797 ft. long and 226 ft. high.

"Forty miles of track have been relaid with 85-lb. steel, replacing 66-lb. and 72-lb., and ballast has been renewed on 100 miles.

"The branch line in the Bitter Root Valley has been extended 8 miles."

Ore Reduction Works.

In the metallurgical field, the metallurgist is keeping far to the front in his practical and scientific application of new methods, new principles and ideas, revising old methods and demonstrating his ability to successfully treat some ores sent him from the mines so low in metallic values as to baffle all physical tests of the mine foreman, one of whose more or less laudable ambitions is to hoist more tons — of something — than does his neighbor.

At Anaconda, in the Washoe Reduction Works of the Amalgamated Copper Company, new processes, improvements and extensions have taken place on a scale tremendous in the magnitude of its economic savings.

Utilization of Waste Heat. — Impressive is the contemplation of the figures representing the energy developed by the utilization of reverberatory furnace heat in producing steam of approximately 4 000 h. p. To the 50 ft. by 20 ft. furnaces, one 300 h. p. Stirling boiler is attached, and to the new 102 ft. by 19 ft. furnaces are two 375 h. p. Stirling boilers set tandem.

The 4 000 h. p. thus developed are used in the compressor plant known as the Smelter Power House. The utilization of this hitherto waste heat means, in money, an average of $750 in 24 hr.

The smelting efficiency of the furnace is unaffected by this arrangement and no alterations were made in the furnace itself; it was, however, found necessary to remove all baffle plates in the boiler.

Briquetting Machines. — The successful use of briquetting machines on a large scale is a new feature in the Washoe Works.

A mixture of a 3 per cent. copper, common looking, dark mud (slums from the slum ponds), and the flue dust from various sources, with 4.07 per cent. coke, passing uninterruptedly from a hopper through a Chambers Bros. Company's (Philadelphia) briquetting machine, finds, in an almost incredibly short time, a comfortable resting place in the blast furnace. Eight pounds each they weigh, and in November last, 458 tons per day of twenty-four hours was the output, saving in money daily $250.

Utilization of Coked-Coal from Reverberatory Grates. — The installation and complete success of the briquetting machine created annually a new use for thousands of tons of small-sized coke. To supply this need in the most economical way, the management turned to the ash-heap, and the "Coking-Plant"

sprang into life, to act, in an obscure corner of the smelter stage, its designated part.

An unimposing little plant it is, down in a corner of the smelter yard; it makes, in its apparent insignificance, the humblest pretensions, entirely out of proportion to the degree of regal importance to which the aggregate value of its product entitles it. The duty imposed upon it is to jig and separate by water coke obtained from the unburned carbon of the furnace ashes, coked-coal that drops through the furnace grates, refuse hitherto consigned to the ash-dump.

In 24 hr., at present, 22 tons of coke are recovered at a cost of 90 cts. per ton, and later this will be increased to 30 tons dry weight. Its product represents a net value of $7.60 for every ton.

Here is the average analysis, in percentages, of the coke thus obtained for the month of November last, on dry samples: Ash, 12.2; volatile material, 37.7; fixed carbon, 48.4; moisture, 1.7.

Blast Furnace. — Increase of size of blast furnaces has always been the tendency in Montana practice, and at the Washoe Plant this year is being constructed one of over three times the capacity of those hitherto in successful operation.

The following table shows some of the data of the two sizes:

Furnace.	Old.	New.
Horizontal section at throat,	72 by 180 in.	72 by 612 in.
Horizontal section at tuyeres,	56 by 180 in.	56 by 612 in.
Charge (ore and flux) weight, lb.,	9 000 to 10 000 lb.	36 000 to 40 000 lb.
Charge, tons in 24 hr.,	452 tons average per furnace, November, 1904.	Approx. 1 600 tons.
Men in 24 hr.,	24, including all classes; charge loaders, coke wheelers, miscellaneous labor, as well as actual furnace attendants.	Of all classes, 60.
Coke, per cent. of charge,	9 per cent.	Approx. 8.6 per cent.
Blast, cu. ft. per ton of ore,	45 000 cu. ft. per ton of charge, or 100 000 cu. ft. per ton of raw ore based on blower displacement.	45 000 cu. ft. per ton charge, or 100 000 cu. ft. per ton ore.
Forehearth, shell dimensions,	16 ft. diameter. 5 ft. deep.	Two 16 ft. diameter. 5 ft. deep.
Tuyeres, number,	26	90
Tuyeres, diameter,	4 in.	4 in.

The water jackets of the new large furnace will be of the same general style as in the present ones — sectional jackets, sheet steel throughout, with "T" iron stiffeners riveted to the back plate instead of stay-bolts.

It is the expectation of the manager that the same character of ores, as regards fines, will be treated in the new furnaces as are in the old.

Reverberatory Matting Furnace. — The old Welsh furnace, with hearth 15 ft. long by 9 ft. 8 in. wide, used in the United States in 1878, of pear-shaped form and a hearth area of 4.67 times that of the fire box, gradually underwent a change of form to the rectangular and with an increased hearth area, until, in 1894, the reverberatory had a hearth area of 14.8 times that of grate area.

The ratio of hearth to grate area of the present Montana furnaces with hearths 50 ft. long by 20 ft. wide, is 15.6.

At the Washoe Works, the furnaces have been increased in the past year to the unprecedented length of 102 ft. (one furnace 112.5 ft.), with a maximum width of 19 ft., maintaining practically the same ratio, 15.8, of hearth to grate.

These large furnaces are completely successful; the special advantages being increased capacity, with savings in coke, repairs and labor.

The following table shows some relative partial data of a Montana furnace and the Washoe Works new furnace:

	Old.	New.
Length of hearth,	50 ft.	102 ft. (1 furnace 112.5 ft.)
Width of hearth, maximum,	20 ft.	19 ft.
Hearth area,	842 sq. ft.	1 767.7 sq. ft.
Length of grate,	10 ft.	16 ft.
Width of grate,	5 ft. 5.5 in.	7 ft.
Grate area,	53.9 sq. ft.	112 ft.
Ratio, hearth to grate area,	15.6	15.8
Charge, weight in tons,	25	15
Charge, tons in 24 hr.,	112.5	250 average; 350 maximum
Rate of concentration,	5.7 : 1	4.5 : 1
Tons of charge to 1 ton of coal,	3.05	4.67 average
Increase of labor in increasing size of furnace,		None

Whether or not the limit in size of the furnaces has been reached, I don't know; but to me, in a recent visit to the Washoe plant, it seemed that the controlling consideration as to the length of the furnace was the size of the previously constructed building within which it was originally planned that reverbera-

tories should be placed. Certainly the year 1904 is distinguished by a remarkable event in reverberatory-furnace practice.

Other improvements at the Washoe Works are planned, and others are present facts; among which might be mentioned the substitution of siliceous gold ores for barren silica in converter linings. Activity for betterments and expansion in all parts of the plant are the striking characteristics of these works.

Impressed is a visitor there with the economic contrivances everywhere utilized; with the orderly, convenient arrangements and the ingenuity and foresight of the original designers of this greatest copper smelter.

The most impressive fact of all, flowing from which are the augmented benefits to the community and state, is that the minimum limit of profitably worked low-grade ores is being brought lower and lower, increasing, by years, the life of a mine and made possible by the progressive spirit of betterments invading all departments and the centralization of operations in mammoth plants.

To Mr. E. P. Mathewson, manager of the Washoe Works, we are indebted for kindly furnishing all data used above in connection with those works.

Great Falls Smelter. — At the Great Falls Works of the Boston & Montana Company, Mr. C. W. Goodale, manager of the company, writes that, during the year, no noteworthy additions have been made, and mentions the following improvements.

"In the concentrator we have installed automatic feeding arrangements at the crushers, with sorting belts. From these belts the ore of smelting grade is picked off and sent directly to the blast furnaces and losses in treatment are thereby reduced. No extra labor is required for this work, as the man who formerly fed the crusher now does the sorting, and the efficiency of the whole mill is better on an automatic and regular feed.

"Settlers have been installed at the reverberatory furnaces, of the same form and with the same lining as those used at the blast furnaces — 16 ft. outside diameter, 13 ft. inside and 60 in. deep. The furnaces, with hearths 42.5 ft. by 15 ft. 9 in., discharge slag and matte into these settlers, and the losses of copper in slag are less than when the furnaces were skimmed. By the use of an automatic and continuous feed of calcines and discharge of slag, the time formerly required for charging and skimming is cut off, and the daily furnace capacity is increased. Longer furnaces will probably be built in 1905, so as to obtain

greater capacity, but with an average duty of more than 200 tons per day, it will be noted that the present 42.5 ft. furnaces are doing good work. It should perhaps be mentioned that in the use of gas in reverberatory smelting, the Great Falls plant differs from all the other copper works of the West. The coal supply comes from Sand Coulee and Belt and is of such a quality that 'direct firing' would not be satisfactory.

"The converters at Great Falls are of the 'upright' type, and the diameter of the bowl is 7 ft. The average duty of one lining is 15 tons of copper. A larger converter, of elliptical cross section of bowl, has been built — longer diameter 10 ft., shorter 9 ft. — and so lined that when filled with matte to a depth of 26 in. above the tuyeres, it will hold twice as much matte as the original converters for the initial charge. It is expected that enough lining material can be placed in this bowl to provide for a duty of 30 tons and that the time required to 'blow' this amount of copper will be no greater than with 15 tons in the old converters."

The Pittsmont Smelter.

Untrammeled by tradition and the practice of copper metallurgists the world over, the Pittsburgh & Montana Copper Company, under the management of Mr. Ralph Baggaley, has erected a reduction plant of new and original methods and of radical differences, astounding to those who are producing copper under the principles and methods hitherto considered essential and indispensable to the successful extraction of the metal from ores of the character of the Butte district.

At this plant, the use of water concentrators, roasting furnaces and blast furnaces, is held in complete contempt and disdainfully disregarded.

All that it is intended to use here for turning out copper, is a converter of special pattern, into which the raw ores, in the condition as they come from the mines, are placed and covered with a coating of molten slag or matte, which, on cooling, cements the material to the converter walls. Then a small portion of matte obtained from what is known as the retreatment furnace, where ores in lump form and slag are melted together, is run in on top of the charge and the blast turned on, which, in a series of experiments in February and March of the past year, reduced the charge in 55 min. to blister copper.

In place of water jackets to protect the converters, which by the continually running water extract and reduce the internal heats, it is intended, in the new 16-ton converter now being con-

structed, to use magnesite brick of a thickness of 9 in. and steel blocks 12 in. thick.

The principle of the process is the melting down and reducing to metal, raw ores, by the combustion of such fuel alone as the ores themselves contain — principally sulphur and iron. If the ores to be treated contain too little of such fuel, lumps of other ores, high in sulphur and iron, are fed to the charge.

The query at this plant is, Why should a metallurgist, by roasting furnaces, devote every effort to the elimination of sulphur when it is the very thing that is needful as fuel, and when its place later is filled by the same thing — fuel — in the form of coal and coke? And why should the silica, by water concentration, be separated and cast away, and silica mines operated to furnish silica in the later stages of reduction?

Since the experiments of a year ago, the company has been making extensive preparations to start up for continuous operation and that will probably be within a short time.

Work of the U. S. Geological Survey.

Topographical. — In course of publication is the topographical map representing the extension to the special map of the Butte district. The original area of the Butte map was about 23 sq. miles, surveyed first in 1895. This area was increased about a year ago by 6 or 7 sq. miles. This extension was to the east of the area originally mapped and includes the "East Ridge," the continental divide and the southernmost point of Elk Park, thus showing the relation of the Butte mining district to the great fault scarp of the main mountain range.

Mr. E. M. Douglas, geographer in charge, writes me that this revised edition of the Butte folio will be published shortly.

The standard map work progressed during the season just passed. It was carried on for the direct benefit of two divisions of work, one for the engineers of the Reclamation Service and the other for the officers of the Forest Reserves. Of the Reclamation Service work, I shall speak later.

The Forest Reserve area mapped was in the northern part of the Lewis and Clarke Reserve. adjacent to the Canadian boundary, between it and the Great Northern Railway and lying west of the main crest of the Rocky Mountains. It embraces the valley of the North Fork of the Flathead River, which is, supposedly, the " Oil Field " of Northern Montana west of the main range.

Of the sliding down hill with an unknown objective point, if any there be to bump up against, of the world's greatest mining

camp, Butte, you have occasionally read in your daily papers. Foundations of houses, churches and in one case, even a millionaire's residence, have, according to these reports, been disturbed to such an extent that perilous was it to tarry longer in the city.

That this was caused by the extensive underground openings of the mines, was the popular explanation of this phenomenon. It is true that, locally, ground has settled and sunk from the surface in measurable extent in places where underground mining operations have been carried on to the exhaustion, near the surface, of the ore bodies, and this has been expected and provided against where necessary. The localities of which we are now speaking are in the immediate vicinity of the mines, to the north and east of the city, but, on the "West Side," where many of us know that there are no underground openings whatsoever, slight cracks in brickwork, in one instance that of a brick church building, are outward signs that some movement has taken place.

In times past we know that great, and many times repeated, earth movements have torn and racked this region; fissure after fissure was opened and movements along fault planes, great and little, took place, and probably are going on to-day. These disturbances we all know were essential to the later mineralization of the fissures that are now being mined.

At one time, the geologists tell us, Meaderville, the east suburb of Butte, was 1 000 feet above the tops of the main range of the Rocky Mountains to the immediate east of it.

They further tell us that these fault movements are still going on about us and that is the reason that there is a crack in the brick wall of the senator's residence and that it was not caused by mining operations.

In the hope that some knowledge might be acquired as to the theory of movements along existing fault planes of the present day and accompanying or following them, the theory that the mineralization of this district is now going on and has never ceased, the topographical branch, during the past season, undertook, at the request of scientific men of Butte, some interesting special spirit-level work.

An east and west line of levels was run from Missoula Gulch to Elk Park, and a north and south line from Timber Butte to Walkerville.

Some 23 bench marks were established covering the city and environs. The levels were run with every precaution to insure

accuracy, and check lines were run to prove the correctness of the work. The regulations of the topographical branch require that the limit of error in feet should not exceed 0.05 $\sqrt{\text{distance in miles}}$. It is planned that these lines be re-run in later years, to determine the character and extent of movements.

Mr. R. H. Chapman, of the Geological Survey, kindly furnished me with the preceding matter, and to Mr. E. M. Douglas, chief geographer of the Geological Survey, I am also much indebted.

Work of the Reclamation Service.

Of the greatest importance to the material welfare of the state is the work now going on under the direction of the U. S. Geological Survey by a division of the hydrographic branch known as the "Reclamation Service."

The Reclamation Law. — The Reclamation Law, approved June 17, 1902, provided in general that certain moneys derived from the sale and disposal of public lands in 16 states and territories of the West should be set aside and appropriated to be known as the "reclamation fund," to be used in the examination and survey for, and the construction and maintenance of, irrigation works for the reclamation of arid and semi-arid lands in the said states and territories.

The Secretary of the Interior is directed to make examinations and surveys for, and to locate and construct such irrigation works.

The Secretary is further authorized to withdraw from entry all public lands which, in his judgment, may possibly be required for any certain irrigation project under consideration, returning the withdrawn lands to entry if the project is determined to be inadvisable.

Upon the determination of the practicability of a project, he shall construct the same, and give public notice of the lands irrigable under such project and limit of area per entry, which limit shall represent the acreage which, in the opinion of the secretary, may be reasonably required for the support of a family on the lands in question; the size of the tract is to be not less than 40 or more than 160 acres.

He also fixes the charge which shall be made per acre upon the said entries, as well as upon the lands in private ownership which may be irrigated by the waters of said irrigation project and the number of annual installments, not exceeding ten, in which such charges shall be paid.

The charges shall be determined with a view of returning to the reclamation fund the estimated cost of construction of the project.

In construction work, 8 hr. shall constitute a day's work and no Mongolian labor shall be employed thereon.

The entryman, in addition to compliance with the homestead laws, shall reclaim at least one half of the total irrigable area of his entry for agricultural purposes, and before receiving his patent shall pay the government the charges apportioned against such tract.

The commutation provisions of the homestead laws do not apply to entries made under this act. If the annual installment extend for 10 years and if it should be decided that this cannot be commuted or paid in advance, then final passage of title may be deferred for upward of 10 years.

At the expiration of the minimum of 5 years, or the maximum of 10 years, it is probable that the persons entering the land will have made permanent homes, the aim being to secure actual settlement and reclamation of the land in small tracts. The law provides that no right to the use of water for land in private ownership shall be sold to any one landowner for a tract exceeding 160 acres.

When the payments are made for the major portion of the lands irrigated, then the management and operation of the irrigation works shall pass to the owners of the lands irrigated thereby, to be maintained at their expense under such form of organization and under such rules and regulations as may be acceptable to the Secretary of the Interior, but the title to and the management and operation of the reservoirs and the works necessary for their protection and operation shall remain in the government until otherwise provided by Congress.

The Secretary is authorized to acquire by purchase or by condemnation under judicial process, any rights or property that may be necessary.

He is further instructed, in carrying out the provisions of the reclamation act, to proceed in conformity with state laws, and he cannot interfere with vested rights acquired thereunder; but there is an important declaration in the act, that the right to the use of the water acquired under the provisions of the act shall be appurtenant to the land irrigated, and beneficial use shall be the basis, the measure and the limit of the right.

It has been found that in some of the states, changes in the laws must be made before important projects can be undertaken.

The state of Nevada has already recognized the importance of action and has passed laws intended to supplement the reclamation law and to assist in carrying out its purpose.

As to the distribution of expenditures, the law virtually divides the fund from each state and territory into two portions, known as the major and minor parts. The expenditure of the major part, which must, of course, be over 50 per cent., is limited to the state or territory within which the money originates, subject, however, to the existence of feasible irrigation projects. The minor portion is unrestricted and can be expended in any one of the 13 states and 3 territories.

The unrestricted funds are to be restored as soon as practicable, and in any event within each 10-yr. period after the passage of the act.

The law covers the sinking of artesian wells and the power development of waters, thereby making necessary the installation of pumping plants in projects where such work will result in reclaiming arid lands.

Petitions from nearly every county and town in portions of the semi-arid regions, asking that artesian wells be sunk, were at one time directed to the Secretary of the Interior and by decision of March 3, 1903, he holds that the reclamation fund cannot be used for drilling artesian wells for exploration. Such wells may be paid for from the reclamation fund only in cases where there is sufficient knowledge in advance to make it probable that water will be obtained therefrom in such quantities as could be used for the irrigation of lands, with the probability that the cost of the work will be returned to the reclamation fund.

Methods Adopted by the Reclamation Service. — A comprehensive set of instructions and suggestions is issued by the chief engineer of the Reclamation Service for the benefit of the engineers in charge of irrigation work, although the engineer is allowed the greatest latitude in the exercise of his judgment. They are in the form of a "Report on Methods of Reconnaissance and Survey," by a committee composed of Jeremiah Ahern, Cyrus C. Babb and J. H. Quinton.

The following extracts are from it:

In spirit-level work, the limit of error in ft. should not exceed 0.05 into the square root of the distance in miles. This is a rule of the topographical branch. The following limiting values for velocity are given, but are not absolute:

2 to 3 ft. per sec. for ordinary soils.

1.75 ft. per sec. minimum, unless water is very clear, when a lower velocity may be used.

3 to 3.5 ft. per sec. in ordinary firm soils.

5 to 7 ft. per sec. are safe in hardpan and soft rock.

8 to 10 ft. per sec. in cement work.

15 ft. per sec. in solid rock.

The velocity and the quantity of water to be carried being fixed, the area of the cross section is known. This area may be put into many different shapes, varying with the material. In the simplest case, the area being a rectangle, there are but two factors to consider, — the breadth and depth.

In trapezoidal channels, an additional factor has to be considered, viz., the slope of the sides. In the most general case, if

A equals the area,
B ,, ,, base,
D ,, ,, depth,
M ,, ,, ratio of slopes, $\frac{\text{horizontal}}{\text{vertical}}$,

The area equals $BD + MD^2$.

We have here three variable quantities, and it becomes necessary to fix two of them before the third can be found. It is evident that M is fixed between narrow limits on account of the material and that B and D may vary considerably. As the width of the base is of minor importance compared with the depth of the water, it is best to fix the latter as soon as the velocity is decided upon.

The depth of the water depends, to a great extent, upon the purpose of the canal. If it is to be an irrigation canal, from which many branches may be taken, the depth should be less than for a canal which is meant to carry flood water to a reservoir. For rectangular channels, one in which the base is twice the depth gives a maximum velocity for a given gradient. This cross section can sometimes be used in flumes and rock cuts, but it is seldom practicable to construct a canal with the most economical form of trapezoidal section, as the slopes are fixed by the nature of the material and the depth of water by other considerations.

In general, the following limiting values for side slopes of canals will be found safe:

½ horizontal to 1 vertical in rock cuts.
1 ,, ,, 1 ,, ,, firm soil.
1½ ,, ,, 1 ,, ,, ordinary soils.
2 ,, ,, 1 ,, ,, very light soils.

On steep side hills the channel should be all in excavation. In level or slightly rolling country, the cost of excavation may be much reduced by making the depth of the cut such as to allow the upper portion of the canal to be made with the material taken out of the excavation.

Slides on the lower side of the canal are apt to occur where the ground on which the embankment rests is sloping, as water is liable to percolate between the new material and the original surface of the ground.

Sometimes a berme of 2 or more ft. is introduced between the foot of the bank and the edge of the excavation, but this is liable to introduce irregularity in the flow of the water if the ground is sloping laterally and undulating longitudinally. If a berme is graded off to true planes, in order to make the cross section conform as nearly as possible to the theoretical one, the additional cost is not justified. Bermes have been very generally discarded in recent irrigation practice. All structures are to be of a permanent nature and of imperishable material.

Montana Projects. — Montana, with its enormous extent of arid land, offers a number of opportunities for reclamation.

Upon a basis of 51 per cent. restricted and 49 per cent. unrestricted, there were in the reclamation fund of Montana for years 1901 to 1903 inclusive, these amounts, that of 1903 being approximate: Total for 1901–2 and 1903, $1 324 546; restricted, 51 per cent., $675 518; unrestricted, 49 per cent., $649 027.

Mr. Cyrus C. Babb, engineer, under F. H. Newell, chief engineer of the Reclamation Service, is in charge of the Montana work of examination and survey and location and construction of irrigation projects.

The land office map of the state shows, where colored in green and blue, the areas withdrawn from entry. Green areas are of lands withdrawn that, it is expected, can be irrigated by projected systems. They are now open to settlement under the homestead laws and the provisions of the reclamation law.

Blue areas are mainly for irrigation works, such as reservoir sites, and are withdrawn from any entry whatsoever.

Milk River Project. — Previous to the passage of the reclamation act, certain preliminary work had been done on what is known as the St. Mary-Milk River Project. This project, briefly, contemplates the storage of water in St. Mary Lake, the carrying of the water in an artificial channel to the headwater streams of Milk River and finally the utilization of the waters in the lower Milk River valley. The first examination and

reconnaissance was made in 1900, followed during 1901 by a detailed survey of a canal line from St. Mary Lakes eastward for 200 miles.

The investigation has brought out three methods of utilizing the water of the St. Mary basin:

1. Diverting the St. Mary River to the north fork of Milk River and allowing it to run through Canada to the lower Milk River valley in Montana.

2. Utilizing the waters on the eastern section of the Blackfoot Indian Reservation and lands immediately adjacent to the east.

3. Carrying water from the head of St. Mary River across both the north and south forks of Milk River to Cut Bank Creek, down which it will flow to Marias River, 100 miles or more, and taking it out of the Marias by a canal to Big Sandy Creek, a tributary of Milk River. The most feasible project is the one first mentioned, but there are a number of international questions connected with it which have yet to be settled.

During 1901 and 1902 attention was chiefly given to surveys in the St. Mary basin. For the last two seasons work has been confined largely to investigations in the lower Milk River valley, where it is proposed to utilize the stored waters. The work comprised detailed surveys and diamond drill borings at a number of dam sites; plane-table surveys of possible canal locations; topographic surveys of agricultural lands.

The entire project has been subdivided into three sub-projects, known respectively as St. Mary Sub-project, Marias Sub-project and Lower Milk River Project.

St. Mary Sub-project. — It is proposed to build a low storage dam about three-fourths of a mile below the present outlet of lower St. Mary Lakes. The dam will have a maximum elevation of 50 ft. above the bottom of the river and will form a reservoir with a capacity of 250 000 acre-ft. The top of the earth dam will be at an elevation of 4 513 ft. above sea-level. The total length of earth embankment will be 2 650 ft. With an inner and outer slope of 3 to 1, the total content of the dam will be 585 864 cu. yd. A core wall will probably be built of concrete at the eastern end, and of puddled earth at the western. The estimated cost of the dam and work is $250 000.

From the reservoir thus formed the water will be diverted through an open cut in the right bank of the river, the formation of which is a fairly durable sandstone. The flow of the water will be controlled by gates placed in a circular cement wall of

30 ft. radius. The canal proper will begin at the end of the open cut. The grade was assumed at 0.02 per cent., or fall of 1.056 ft. to the mile. It is proposed to make the bottom width 30 ft. with a 10-ft. depth of water and with side slopes of 1 to 1. It is assumed that in a canal of these dimensions, the mean discharge at the head will be 1 400 sec.-ft. From the dam the canal will continue down the east bank of the river a distance of 7 miles, then turn eastward and pass through what is known as Spider Lake Gap. This gap is a natural depression in the high Milk River ridge. If it were not for its occurrence, a tunnel would have been necessary about 2 miles in length. Thence the canal continues in a general northeasterly direction to the north fork of the Milk River, the total distance to this point from the head of St. Mary River being 27.4 miles. The last step in the construction of the canal to reach the north fork will be the dropping of the water 180 ft. to the level of the creek.

The estimated cost of the storage of the St. Mary basin and the construction of the canal to the north fork is $924 070.

Marias Sub-project. — The diversion of Marias River has, previous to this year, been considered more as a connecting link between the St. Mary basin and the lower Milk River valley, but from developments this year the link is now being considered as a separate project to be developed on its own merits.

During 1902 detailed surveys were started by a thorough examination through the headwaters of Marias River, looking for possible reservoir sites. Several feasible ones were found, but not of as large capacity as was desired. The most notable one is that of Two Medicine Lake. A dam 90 ft. high immediately below the outlet of the lake would store 87 600 acre-ft. of water. The length along the top of such a dam would be 2 150 ft.

During the same year a topographic map of the Marias River canyon on a scale of 0.20 mile to the inch and with a contour interval of 20 ft., was made from the crossing of the Great Falls & Canadian Railroad to below the mouth of Cottonwood Creek. During the present year, 1904, investigations have been carried on more in detail. The topographic survey of the canyon was continued to below the big bend of the Marias and the most feasible dam site on the river was found. It is at what is known as the Sandstone Canyon. Detailed surveys of this dam site were made on a scale of 100 ft. to 1 in. and the canyon itself below the dam site was also mapped on the same scale in order to give the best location for the diversion canal. Diamond drill borings were

made along the center line of the dam, along the projected outlet tunnel line and across a natural spillway. These borings brought out the fact that a good sandstone rock occurred very close to the surface in most places, not being more than 20 or 25 ft. below the average elevation of the ground. The plan contemplates the construction of an earth dam from 150 to 185 ft. in height with a masonry or concrete core wall. Plans and estimates for this project are now being worked up in the office of the engineers of the Reclamation Service.

It is recognized that the diversion of Marias River is the most difficult problem connected with the Milk River project. In some ways the Marias River has greater advantages than almost any other project in the arid West. The area that can be irrigated from it, from 200 000 to 300 000 acres, is nearly all government land, is located in one very compact area and has a type of topography almost ideally suited for the easy construction of lateral systems of ditches. It is a project that may not be taken up immediately, but will receive a development in the future.

Lower Milk River Valley. — The first plan developed for the utilization of St. Mary water in the lower Milk River valley was what is known as the "Dodson diversion." This plan was to divert water from the south side of Milk River, about 23 miles west of Malta, and carry it eastward for 30 miles to Lake Bowdoin reservoir, at present a small alkali body of water 10 miles east of Malta. The canal to the reservoir was to be a feed canal to carry flood waters. It was to be 50 ft. wide on the bottom, 12 ft. deep and on a grade of about 6 in. to the mile.

In order to create a reservoir at Lake Bowdoin, it will be necessary to build a series of earth embankments across various gaps, 8 in all. The combined embankments would contain 451 783 cu. yd. of earth. The reservoir thus constructed would have a capacity of 299 000 acre-ft. During the last two seasons surveys have also been made for canal lines extending from the Dodson dam eastward as far as Glasgow and including a distribution system from Lake Bowdoin. From these surveys, and from the maps of the agricultural lands, there has been found that the total acreage that can be covered by possible canals under this system is 158 900 acres.

The results of work of this season, 1904, have brought out a better reservoir proposition than the Lake Bowdoin, known as Chain Lakes. This site is partly on Milk River and partly in a series of lakes which gives the name to the site. The main dam

on the Milk River is located about 25 miles northwest of Havre. The plan as at present outlined is for the construction of a dam 90 ft. high on the main channel of Milk River which will create a reservoir of 437 000 acre-ft. capacity.

From this reservoir water would be returned to the main channel of Milk River, down which it would flow for 35 miles to the first diversions near Chinook, where a portion of the water would be taken out, the balance being allowed to continue down to the Dodson diversions.

During 1903 a canal was located, known as the Yantic Canal, heading on the north side of Milk River about 7 miles west of Chinook. This canal would be 54 miles long and control 63 500 acres of land. During the past season a second canal, heading 3 miles southeast of Chinook and taken out of the south or right bank of Milk River, was located eastward for 57 miles. The greater portion of the land under this canal is on the Fort Belknap Indian Reservation and it will be necessary to open this portion of the reservation if the area is to be utilized for white settlement. The total area on the Chinook Canal is 48 000 acres, 33 000 being on the Indian reservation and the balance, 15 000, west of it.

The cost of the Milk River project is estimated at $20 to $25 an acre, payable in ten annual installments without interest, or at the rate of $2 to $2.50 a year for ten years.

Lower Yellowstone Project.

This project, or as it is sometimes known, the Fort Buford Project, contemplates the diversion of the Yellowstone River on the west or left bank, about 20 miles below Glendive, Mont. Surveys were inaugurated on this project in 1903, when a high-line canal was run, located some distance above the one just mentioned. When the estimates were worked up last winter, however, it was found that the cost would be excessive, due to the numerous stream crossings and to many miles of side-hill work. When the surveys were resumed this past spring, attention was given to the low line diversion. The results of the work show that a canal can be built 65 miles in length and covering 60 000 acres of irrigable land. The estimated cost is $30 per acre. Detailed surveys were made of the diversion point on Yellowstone River and many borings at the dam site as well as along the canal location. It is expected that during this coming winter, detailed plans and estimates will have been made so that bids can be asked for the construction of the dam and head gates early in the spring. After that, the work will be rapidly pushed

and two or three seasons should see the completion of the entire construction.

Sun River Project.

Sun River receives the drainage area from the eastern portion of the Rocky Mountain region, south of the St. Mary River and north of the Dearborn. It flows in a generally easterly direction, entering the Missouri at Great Falls, Mont. An examination of this stream was made by the Irrigation Survey in 1890–91, the results being published in the Twelfth Annual Report of the Geological Survey, Part II. There are known to be several reservoir sites in the basin and there is a large body of irrigable land that can be reclaimed at reasonable cost, much of it in public ownership.

During the present season surveys have been actively prosecuted within the area, including location of canal lines from Sun River and a detailed survey of a reservoir site known as Freezeout Basin. The possibilities of this project, including diversions from Sun River and Teton River, are such that probably two or three field seasons are necessary to bring out the most advantageous canal lines. Preliminary designs and estimates of the cost are now being made, to be submitted to the Secretary of the Interior for his favorable consideration.

Crow Reservation Project.

By act of Congress of April 27, 1904, a certain area of the Crow Indian Reservation was to be opened for settlement. Under this act the Crow Indians ceded to the United States the strip of land in the northern part of their reservation, subject to certain conditions, including the payment of $1 000 000 and the surveying of the lands. After the completion of allotments to Indians, the residue of the ceded lands are to be subject to withdrawal and apportionment under the reclamation act. It is provided, however, that if the lands withdrawn under the reclamation act are not disposed of within 5 years after the passage of the act of April 27, 1904, then all of the lands are to be disposed of as other lands provided for in the act. The price of these lands is to be $4 per acre when entered under the homestead law, in addition to the cost of reclamation.

Soon after the passage of the act opening the reservation, a party from the Reclamation Service was placed on the ground. A preliminary reconnaissance was made, which brought out the fact that approximately 200 000 acres were available for reclama-

tion on the reservation, comprised under two or three diversion possibilities.

From Big Horn River a diversion canal heading near Fort Custer will control approximately 40 000 acres. During the past season this line was surveyed in detail and estimates are now being prepared by the engineering force of the Reclamation Service. A second diversion high-line canal is possible, heading at the mouth of the canyon above the one just mentioned, which will cover about 100 000 acres. There are several difficult engineering features, however, connected with this latter project and it was not examined in detail this season. A third possible diversion is from the south bank of Yellowstone River, heading near Huntley, whereby approximately 40 000 acres can be reclaimed. This canal was surveyed in detail during the past season and estimates regarding cost are now being prepared. The line is an eminently feasible one, with few engineering problems, and it is hoped that bids may be asked for its construction during the coming season. The estimated cost has not yet been prepared, but it is believed that it will be below many of the projects already receiving favorable mention.

A fourth diversion possibility has been considered below the mouth of Big Horn River to lands included within the Sanders Flat country. No work beyond the preliminary reconnaissance has been done there, but it has many features that render a further investigation desirable.

A preliminary examination of the lower Yellowstone valley between Glendive, Mont., and Buford, N. Dak., was made in 1903, by F. E. Weymouth; the result of that season's work demonstrated the practicability of the project there of reclaiming about 70 000 acres of land on the west side of the river, nearly all of it being in Montana, as seen from the map.

The work of survey and location was taken up during the 1904 season and the line of the canal, 65 miles in length, was located.

The irrigable land has been classified and mapped to a scale of 1 000 ft. to an in., with 5 ft. contour intervals, so that the sizes of the main canal and the location of the laterals and pumping stations can be determined.

Borings were made at the dam site, at the head gates, in the deep cuts along the canal line and at all creek crossings where masonry structures are to be built, to determine the nature of the material and presence of water. Test pits and auger holes have been put down at frequent intervals along the canal

line, and the nature of the material in the bed of the canal ascertained.

During the winter final plans and estimates are being worked up and the construction work will probably begin about May 1, 1905.

Water Supply. — The national government, through the U. S. Geological Survey, has been studying the water resources of the country since 1888, or since the time of the old irrigation survey, as it was called. Certain gaging stations first located during that period have been maintained to the present date. Every year additional ones are established.

A number of prominent engineers have recently remarked, that if it had not been for this earlier work of the Hydrographic Division of the Geological Survey, the present reclamation construction work now in progress would have had to be postponed for several years.

Montana is fortunate in having some of the earliest stations that were established, and for a few of the rivers from 12 to 14 year records of discharge are available.

The results of stream measurements are published from year to year in the annual reports and water supply papers of the Geological Survey.

There are now being compiled and corrected all back records of the discharge of Montana streams, and it is expected that they will appear in a special water-supply paper during the coming year.

The following is a list of stations maintained in Montana during 1904:

Gaging Stations in Montana Maintained by the U. S. Geological Survey during 1904.

St. Mary River at dam site.
St. Mary River at international line.
Swift Current, near mouth.
Marias River, near Shelby.
Milk River at Havre.
Milk River at Malta.
Sun River at Christian's Ranch.
Missouri River at Cascade.
Jefferson River at Sappington.
Madison River at Norris.
West Gallatin River at Salesville.
Gallatin River at Logan.
Yellowstone River at Livingston.
Yellowstone River at Billings.

Yellowstone River at Glendive.
Pryor Creek at Huntley.
Big Horn River at Crow Agency.
Big Blackfoot River at Bonner.
Missouri River at Missoula.
Bitter Root River near Missoula.
Bitter Root River near Grantsdale.

The following equivalents may be convenient in connection with the irrigation work.

To change miles to inches on maps:
Scale 1 : 125 000, 1 mile = 0.50688 in.
Scale 1 : 90 000, 1 mile = 0.70400 in.
Scale 1 : 62 500, 1 mile = 1.01376 in.
Scale 1 : 45 000, 1 mile = 1.40800 in.
1 cu. ft = 7.48 gal.
1 cu. mr. per min. = 0.5886 sec.-ft.
1 sec.-ft. = 50 California miner's inches.
1 sec.-ft. = 40 Montana miner's inches.
1 sec.-ft. = 449 gal. per min.
1 sec.-ft. for one day = 1.9835 acre-ft.
1 sec.-ft. for one day = 646 272 U. S. gal.
1 sec.-ft. = about 1 acre-in. per hr.
1 h. p. = 1 sec.-ft. water falling 8.8 ft.
1 acre-ft. = 325 850 gal.

UNITED STATES MINING LAWS.

By Charles W. Goodale, Member of the Montana Society of Engineers.

[Read before the Society, January 14, 1905.]

A commission was appointed by President Roosevelt on October 22, 1903, made up as follows: W. A. Richards, commissioner of the General Land Office; F. H. Newell, chief engineer of the United States Geological Survey and Gifford Pinchot, chief of the Bureau of Forestry, with the following duties:

"To report upon the condition, operation and effect of the present land laws, and on the use, condition, disposal and settlement of the public lands . . . and especially what changes in organization, laws, regulations and practice affecting public lands are needed: first, to effect the largest practicable disposition of the public lands to actual settlers who will build permanent homes upon them; second, to secure in permanence the fullest and most effective use of the resources of the public lands; and it will make such other reports and recommendations as its study of these questions may suggest."

The mining laws of the United States are of such importance in connection with the development of the public land resources that the commission would have been given additional strength if a mining engineer had been included in the list of appointments, but the commission has adopted a plan to get opinions and suggestions from those who have had experience in the administration and in the effects of existing laws.

Individual members of the Montana Society of Engineers have been asked to assist in this matter, but there is no doubt that resolutions adopted by the society as a whole would have great weight with the commission. If we can go further than that and frame some amendments to be taken up in Congress, a shorter route to actual results will be opened. There is probably no state in the Union where objections to existing laws have been more forcibly demonstrated than in Montana, and therefore this society, which includes so many engineers with experience in mining litigation, should be able to point out definite remedies. Some of the provisions of our mining laws may be entirely satisfactory, but that feature of the Revised Statutes embodied in Section 2,322, which gives extra-lateral rights to locators of lodes, is not very popular in Montana.

Reviewing published discussions on the subject of our mining laws which have appeared recently, Dr. R. W. Raymond's article in the *Engineering and Mining Journal*, of June 16, 1904, was of great interest, and other engineers in later issues of the same journal have contributed valuable suggestions.

The debate in the United States Senate, December 15, 1904, on the mining laws enacted for the Philippine Islands, gave Senator Heyburn, of Idaho, an opportunity to defend extra-lateral rights, but let us see if there is not a good and sufficient answer to his arguments.

The Philippine law, which was passed July 1, 1902, provided for mining locations 300 m., or approximately 1 000 ft., square, giving rights to all the mineral therein contained, but with no privileges of following the veins on their dip into adjoining territory.

The Senate Committee endeavored to unite the best features of the British Columbia and Mexican mining laws, but Senator Heyburn maintained that the laws as passed would retard the development of the mineral resources of the Philippines and that American capital would not undertake mining operations under such laws. This is certainly refuted by the fact that capital from the United States has eagerly sought mining investments in Mexico and British Columbia, and on the other hand, there is not the slightest doubt that English and other foreign capital would have been invested in our mines in much larger amounts if the dangers of litigation, arising from extra-lateral rights, had not held it back.

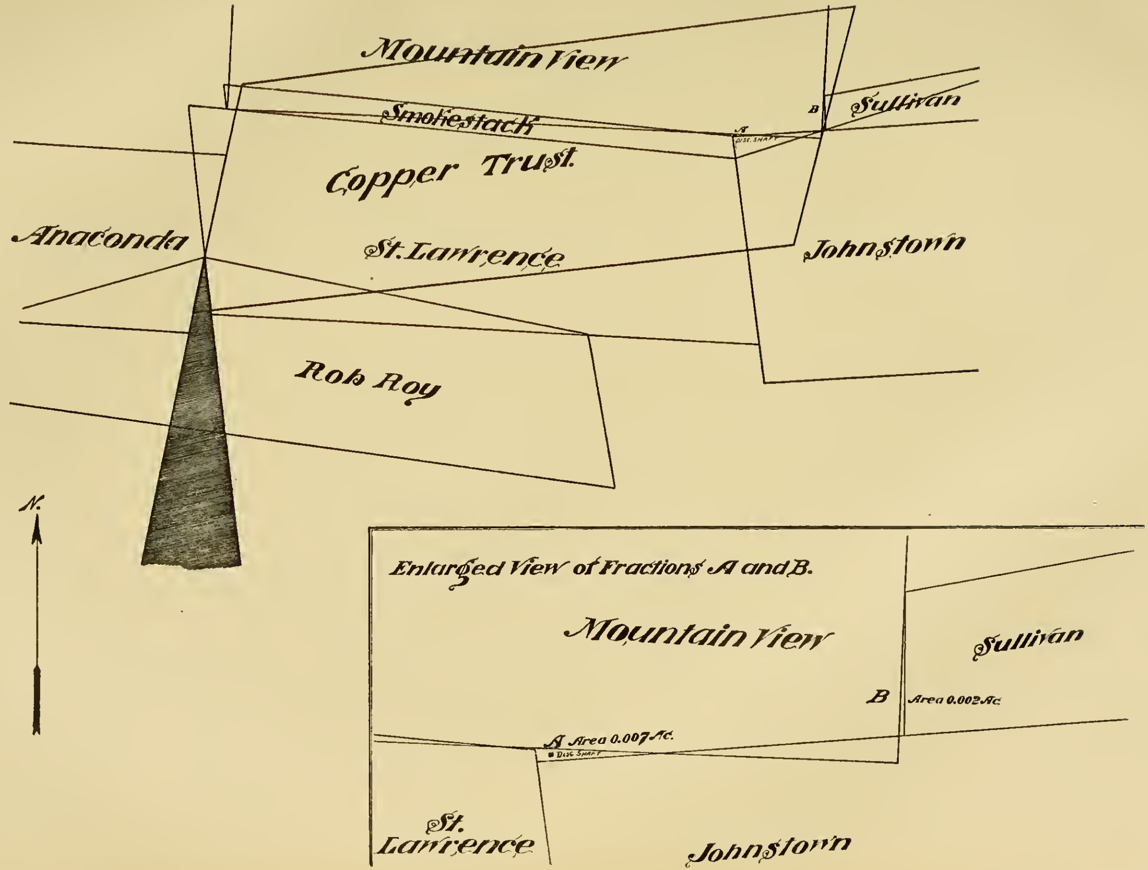
Complaints from American mining operators in British Columbia are not based upon the restriction which says, "The holder of a mineral claim . . . shall not be entitled to mine outside the boundary lines of his claim continued vertically downward," but upon the uncertainties, due to frequent amendments, regarding other requirements of the law. Prior to 1884 there were no laws in British Columbia in relation to lode claims, placer locations alone having received attention in legislative acts of the province; but in 1884 a mineral claim was defined as 1 500 by 600 ft., with surface and extra-lateral rights. In 1891, new provisions were inserted in the law as to record, annual assessment work, etc. On the 23d of April, 1892, the statute was repealed, and a new one enacted, allowing no surface rights except for mining purposes, and doing away with extra-lateral rights. The size of the location was fixed at 1 500 by 1 500 ft. Numerous amendments have been made since that year in regard

to the method of location, number of posts, certificates of annual assessment work, etc., and it is probably true that an ordinary miner would have difficulty in interpreting the present law, but there has been no going back to extra-lateral rights, and mining communities have no desire to do so after a test of nearly thirteen years.

Senator Heyburn intimates that with a United States law giving no extra-lateral rights, a man, after making one *legal* location, could steal twenty more pieces of land from the government by making flanking locations to protect his rights on the dip. How could it be called stealing if, in getting his patent title from the government, he had to pay $5 per acre, the rate now required in patenting any lode location? Under the present law the wise prospector makes side locations as well as "extensions" on the strike of the vein, and if he has not protected himself against litigation before he offers his mine for sale, the prudent buyer does not overlook the possibilities. We all know of instances in this state where individuals and companies hold under locations or have patented large areas around a lode claim and sometimes on rather questionable discoveries of a "well-defined crevice," or of "mineral in place," and we cannot see how a law similar to that in force in British Columbia would give any greater opportunities for the acquisition of mineral lands "by those who want to acquire vast areas of surface ground and deprive the miner of the right to prospect upon it."

Can any one question the statement that a law which made possible the notorious Copper Trust litigation should be revised? The Copper Trust lode claim, 0.009 of an acre in extent, was located upon a triangular piece of ground in the heart of the Butte district and its owner asserted title to a portion of the Anaconda-St. Lawrence vein, this portion having its apex 1 265 ft. from the ground located, and an injunction was issued by the District Court which shut down the Anaconda and St. Lawrence mines, throwing more than a thousand miners out of employment.

The Copper Trust lode claim was located April 30, 1899. The only unpatented ground within the lines of the Copper Trust, as located, consisted of two small fractions, one of which is shown on the map at "A," where the discovery shaft was located, being a triangular piece of ground 10 ft. wide at the base and extending east a distance of 75 ft. and containing an area of $\frac{7}{1000}$ of an acre, being a piece left vacant upon the correction by patent surveys of the north side line of the Johnstown, and the south side line of the Mountain View lode claims; and a


Mountain View
Smokestack
Copper Trust.
Anaconda
St. Lawrence
Rob Roy
Sullivan
Johnstown
B
A
DISC. SHAFT
N.
Enlarged View of Fractions A and B.
Mountain View
Sullivan
B
Area 0.002 Ac.
A
Area 0.007 Ac.
DISC. SHAFT
St. Lawrence
Johnstown

small triangle located at the point "B," containing $\frac{2}{1000}$ of an acre, which lay between the west end line of the Sullivan and the east end line of the Mountain View as surveyed for patent. With the exception of these two tiny fractions, all of the ground embraced within the Copper Trust location, or with which it conflicts on the surface, or the extra-lateral rights claimed for the Copper Trust, had been, for many years prior to the Copper Trust location, patented ground. The Copper Trust owners contended that their discovery was made upon what is known as the Anaconda vein, which extended through their point of discovery and west through the west end line of the St. Lawrence lode claim and the east end line of the Anaconda lode claim.

That the owners of the Anaconda and St. Lawrence lode claims, being confined to the planes of the east end line of the Anaconda and the west end line of the St. Lawrence, respectively, in following said vein upon its dip to the south, acquired no rights in the said vein, after the same on its dip passed through the said end lines respectively and entered the apex of the triangle shown on the map in hatched lines, and that under the exclusions contained in their patents the owners of the Rob Roy and other claims to the south obtained no rights in said vein, as its apex was without their ground, and that, therefore, there was left a segment of this vein shown by the triangle in hatched lines upon the map, which increases as the vein descends into the ground to the south, upon which neither the Anaconda nor the St. Lawrence nor the Rob Roy nor any one but the government had any rights at the time that the Copper Trust was located and that by this location the Copper Trust acquired, not alone the surface rights in the two small triangular pieces upon which the location was based, but also acquired the segment of the vein lying within this triangle east of the plane of the west end line of the Copper Trust extended in its own direction to the south.

This contention was fully sustained by the District Court of Silver Bow County, but the Supreme Court of Montana repudiated the claim, holding that the Copper Trust acquired nothing but the rights given by the two triangular fractions of unpatented ground included within the location.

This is only one example of the dangers incident to our present law, but many others could be mentioned.

With a law in force giving no mineral rights outside of vertical boundaries of the ground located, controversies between adjoining claims could be settled with mathematical precision,

but under our law, mining cases frequently require many weeks for trial, and "lawsuit geology" and theories are explained to judges and juries who are utterly unable to understand the statements made and the maps and models exhibited. Many of us know that hundreds of thousands of dollars have been spent in this district alone in underground law-suit work, or in "war measures." Then again, after large ore bodies have been opened up, the *prima facie* owners have been enjoined from working them because the claim adjoining asserted adverse rights, and while waiting for the "law's delays" to settle questions of ownership, workings have caved in and ore and waste have become so hopelessly mixed that very little, if any, profit can be realized when title has at last been quieted.

Senator Heyburn says that the present law "gives a man a definite estate which no man can take away from him," but, on the contrary, is it not true, as Dr. Raymond contends, that there is nothing definite about the rights granted by a United States patent?

If locations were always properly made, if veins were not variable in dip and strike, if intersections or unions did not occur, if there were no faults and if expert witnesses could agree as to vein definitions, owners of adjoining claims could settle their disputes and there would be no uncertainty regarding the rights granted by a United States patent. The only wonder is that a law so fruitful in litigation in nearly every mining district of importance in the United States has been allowed to stand for more than thirty years.

Dr. Raymond and other writers have called attention to a defect or omission in our law which is very important and should be remedied,—the federal government should have prompt notice of all locations on its public lands. Where locations are made on unsurveyed lands, the records would not be very satisfactory, but the locality could be described with sufficient accuracy to decrease the chances of conflict between agricultural and mineral claimants when the surveys are extended.

In considering the question of a revision of our mining laws, a brief review of some of the conditions in the laws of other countries would be of assistance. The principal features of the mining laws of British Columbia may be stated as follows:

First. Any man desiring to prospect for minerals takes out a free miner's certificate, paying therefor $5 per year, and the privileges under this certificate expire on the 31st of May of each year. This certificate is not transferable.

Second. This gives the right to locate minerals other than coal.

Third. The dimensions of a full claim may be 1 500 by 1 500 ft., but fractional claims are permitted. The locator is entitled to all the minerals which may lie within his claim, but he has no right to mine outside of his boundaries.

Fourth. The holder of a free miner's certificate is not permitted to locate more than one claim on the same vein or lode, but he can acquire, by purchase, additional claims on the lode which he has located. He is allowed to take up claims on separate veins or lodes.

Fifth. The surface right is only acquired for the purpose of mining; all other surface rights are vested in the Crown.

Sixth. Record must be made within 15 days with the mining recorder if the location is within 10 miles of the office of said mining recorder. One additional day is allowed for each 10 miles of distance. The record must give the name of the claim and of the locator, also the number of the free miner's certificate under which the claim is taken up; locality, direction of location line, length and date of location must also be specified.

Seventh. Location holds for one year, $100 worth of work required on each location after that, and an affidavit of this work must be filed with the recorder.

Eighth. If work to the extent of more than $100 is done in one year, such excess will apply to succeeding years by paying the recording fees for the additional affidavits.

Ninth. A payment of $100 to the mining recorder takes the place of assessment work.

Tenth. After doing $100 assessment work, or paying $100 as above provided, and application has been made for certificate of improvement, no more work is required.

Eleventh. No relocation of an abandoned claim is allowed without first obtaining permission from the gold commissioner, and a fee of $10 is paid for such permission.

Twelfth. Crown grants are issued for mining locations after work to the extent of $500 has been done, or after a payment of this amount has been made to the mining recorder. If less than $500 worth of work has been done, the locator can obtain a Crown grant by paying the difference between the value of work performed and $500.

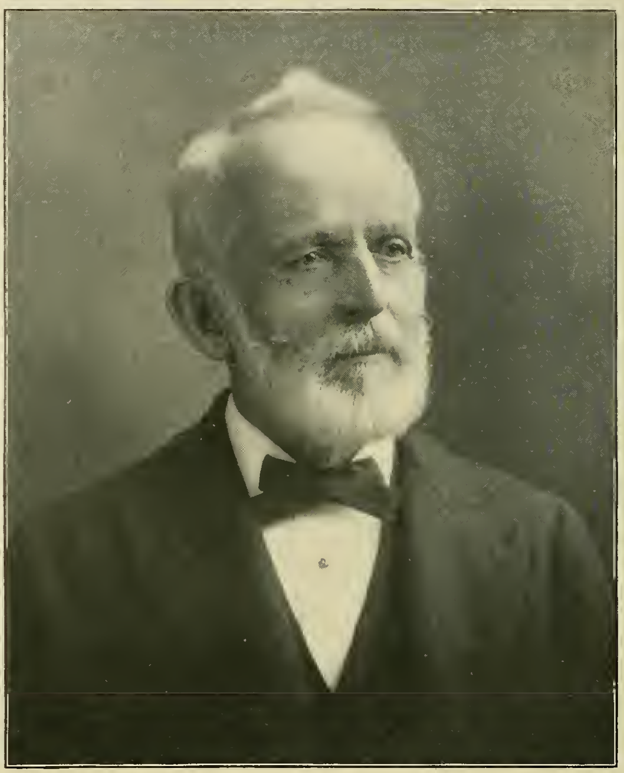
Without going into full details regarding the Mexican mining law, the following notes will show what is required in that republic. A mining claim is called a *pertenencia*, and is

100 m. square, embracing an area of 2.47 acres. A notice of location, or, as it is called, "denouncement," is filed in the office of the Mineria and from there the notice is sent to the federal office in the city of Mexico.

No discovery of mineral within the claim is required. Patents are not given for mining ground, but owners of claims are compelled to pay $10 a year in taxes to the government. This may be all paid at once, or divided into three installments, but on default of the third payment the claim is subject to relocation.

In West Australia claims may be staked out in any shape or size, but not in excess of 24 acres, nor can they be over twice as long as they are wide. Application for registration must be made practically immediately and the government sends a surveyor to correct the location very soon after it has notice and at charges which are very moderate. The claims thus obtained carry all ores within vertical boundaries. Litigation over title is unknown. The tendency in Kalgoorlie has been to systematically develop a claim whether there were favorable indications at the surface or not, as the title to all ore found within the claim boundaries was assured.

The administration of all matters pertaining to mining is placed in the hands of a warden appointed by the Crown. They act as judge in all cases of mining questions, such as breach of regulations, "jumping," etc. These cases are tried without a jury, and the inspector of mines is the official source of information regarding the facts, so the mine inspector is really the assistant to the warden.



G. A. Hyde.

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FIFTY YEARS' WEATHER AT CLEVELAND.

BY G. A. HYDE, MEMBER OF THE CIVIL ENGINEERS' CLUB OF CLEVELAND.

[Read before the Club, November 14, 1905.]

I DID not expect recognition by this club of my volunteer services in the cause of the science of meteorology, but I greatly appreciate your kindness and thoughtfulness.

When I volunteered my services in this department of science, it was the beginning of an effort to discover the characteristics of storms that traverse this country, of which very little was then known.

Prof. J. P. Espy, author of the "Philosophy of Storms," desiring to "find out all the *phases*" described in his "Philosophy of Storms," issued a circular "To the Friends of Science," December 6, 1842, asking them to "coöperate with him in his endeavors to find out all the *phases* of storms which occur within the range of the widespread simultaneous observations about to be established in our country."

In response to this call, one hundred and twenty persons, including myself, volunteered their services and commenced, early in 1843, to take the desired observations.

Professor Espy, by the aid of these observations, was able to prove his "theories of storms," and made the first report of his findings to the surgeon-general of the United States army, October 8, 1843.

In a subsequent report he gave further and more minute details of his findings, and also gave the names of those who had assisted him by taking the observations he desired.

Since that date, the same observers, and many others, have continued taking observations and have sent them to the War Department, Smithsonian Institution, Signal Service and Weather Bureau. The investigation of these observations has resulted in the establishment of the present valuable Weather Bureau and the giving to the public daily forecasts of the weather, which are proving of incalculable value to the sailor, farmer and average citizen.

It has always been a source of gratification to me that my name appears among the original volunteer observers, and I believe that I am the only original observer that is now taking observations and sending them to the Weather Bureau. These fifty years of observations, revealing the characteristics of the weather at Cleveland, are of much value to those interested in such matters, or who have occasion to use such data.

Not possessing an anemometer, I am unable from my own observation to give the wind velocities, but through courtesy of the Weather Bureau, I am able to say that the highest velocity attained here since the starting of the Weather Bureau (about 32 years ago) was 73 miles per hr., November 26, 1895, and the average velocity is 13 miles per hr.

The question may be asked, What has been learned by the meteorological investigations that have been carried on since Professor Espy commenced them in 1843? It is shown that there are cyclones crossing our country in various directions, but mostly from the west to the east. They have diameters varying from 1 000 to 3 000 miles. Some come on to the land from the Pacific Ocean, and a very few from the Atlantic, but the greater number are formed in the western part of our country in the region of the Rocky Mountains, and then move eastward on various lines. They have a center with a low atmospheric pressure which increases to the outer limit. They revolve on their centers in a movement the reverse of the hands of a watch, and the air ascends at the center. These cyclones are accompanied by anti-cyclones at a distance of one or more thousand miles, having the reverse characteristics, the center with high atmospheric pressure decreasing to the outer limit and having a rotary movement with the hands of a watch and the air descending at the center. These cyclones in their movement eastward cause east, southeast and south winds on their eastern portion which are measurably warm, and west, northwest and north winds on their western side, which are measurably cool and, followed by an anti-cyclone, bring cold air from above,

sometimes causing great and sudden changes in temperature, called cold waves.

During the summer months tornadoes occur in the central districts of our country, and thunderstorms in all portions. They are, in fact, part of cyclones; are found in the right or southeast quadrant and are caused by the cool anti-cyclonic atmosphere overflowing the warm air of the cyclone, thus causing condensation of the warmer humid atmosphere and the formation of storm clouds.

With the use of the telegraph and systematic methods in applying the knowledge obtained from the investigation of these cyclones and publishing weather forecasts, great benefits have accrued to the lake faring man, the farmer and those who navigate or live near our large rivers, giving warning of approach of great and severe storms, the prospect of great floods and cold waves, by which means lives and property are often saved. Forecasts of fair and pleasant weather are also largely beneficial.

Forecasts by the Weather Bureau are not always realized, but about 90 per cent are: The weather map shows the location of a storm center,— a low,— and the presumption is that it will continue on the same course that it has since the last previous report, and forecasts are made in accordance with that information. From some cause the line of movement of the storm is changed from that anticipated, or the storm comes to a standstill, or moves to the right or left in an entirely different course from that anticipated; consequently the forecast is made void.

There are thousands of old and odd sayings in reference to the weather, some of which are good, but the mass are of no value. From my own experience, I have but little faith in any forecast except that made from information received daily from all parts of our country through the Weather Bureau. This information shows the location of the storm centers, the cyclones and anti-cyclones, and the line of their movement can generally be very satisfactorily surmised by the forecaster.

The uncertainty of correctly forecasting the weather was probably the cause of the following unique reply to a question as to what the weather was going to be: "Oh, it's going to clear off and be cloudy, and we are going to have a long spell of weather."

Not having recourse to the Weather Bureau information and forecasts, a person may obtain much satisfaction by having a barometer and studying its changes, and watching the weather conditions that attach to its rise and fall.

The fall of the barometer indicates the approach of a storm, and a rapid fall indicates a rapid approach and high winds. If a person desires to know approximately the location of the approaching storm, he can make it out by turning his face towards the wind and then pointing to the right at an angle of about 90 degrees; he will very nearly point toward the storm center.

If the barometer has been falling for a time and the wind at the observer is from the south, indicating that a storm is approaching from the west, and then the barometer ceases to fall, showing that the storm is not approaching the observer; then, if the wind gradually changes from the south to southwest and west, it will be evident that the storm has taken a course passing north of the observer; and if the wind gradually changes from the south to southeast and east, it will be evident that the storm is taking a course passing south of the observer.

Some persons attempt to forecast the weather for weeks, months and sometimes a year in advance. I am not prepared or inclined to discuss any of the theories of the long-range weather forecasts. From my own observation, covering over 62 years, I am free to say that I believe that there is very little that can be told of the weather of the future. Just here I am reminded of the story of an almanac maker who forecast the weather for a year and sold his almanacs to farmers and others. He was riding through the country, and seeing a farmer near the roadside, said: "Good morning, Mr. Farmer. This is a beautiful day." "Yes," answered the farmer; "but it's going to rain." The almanac maker passed on and soon after the sky clouded and rain fell. "Well," he said to himself, "I would like to know how that farmer knew it was going to rain. I think I will stop on my return and ask him." So, on his way back, he saw the same farmer and asked him how he knew it was going to rain when it was so fair and pleasant in the morning. "Well," said the farmer, "there is an old fool of an almanac maker that tells what the weather is going to be, and I have noticed that the weather is always opposite from what he says, and as he said that it was going to be fair to-day, I knew it would rain" — and so it did.

The Weather Bureau has investigated long-range weather forecasting, and I will present to you some of its opinions. Prof. Willis L. Moore, chief of United States Weather Bureau, writing about long-range weather forecasting, refers to a certain forecaster as follows:

"His forecast statement for May, 1904, ends as follows:

"'The fifth storm period will be central on the 29th and and there will be violent disturbance. Watch the barometer, and if you have a trembling wife and children clinging to you for protection, provide some place of safety in which to resort in case of danger.'

"Is it possible [says Professor Moore] to soar to greater heights of nonsense? In what particular continent or country will the storm period be central on the 29th? Is the entire population of the United States, or the world, expected to dig cellars or caves of shelter in anticipation of a possible occurrence of a tornado whose path of destructive violence would not cover an area represented on a large map by a mark 0.5 in. in length made with a sharp pencil? Is it possible that a man who issues such totally unwarranted sensational and harmful forecasts is seriously considered by the intelligent portion of the American public?"

Prof. C. M. Woodward, of Washington University, criticising the long-range theories of Mr. John H. Tice, in his "Elements of Meteorology," writes:

"So baseless have we found our author's theory, and so deficient are his arguments, that the book seems hardly worthy serious consideration."

Prof. E. B. Garriott, in his "Discussion of Long-Range Forecasts," says the following:

"1. The systems of long-range weather forecasting that depend upon planetary meteorology; moon phases, cycles, positions or movements; stellar influences or star divinations; indications offered by observations of animals, birds and plants, and estimates based upon days, months, seasons and years, have no legitimate basis.

"2. That meteorologists have made exhaustive examinations and comparisons for the purpose of associating the weather with the various phases and positions of the moon in an earnest endeavor to make advances in the science along the line of practical forecasting and have found that while the moon and perhaps the planets exert some influence upon atmospheric tides, the influence is too slight and obscure to justify a consideration of lunar and planetary effects in actual work of weather forecasting.

"3. That stars have no appreciable influence upon the weather.

"4. That animals, birds and plants show by their condition the character of past weather and by their actions the influence of present weather, and the character of weather changes that may occur within a few hours.

"5. That the weather of days, months, seasons and years affords no indications of future weather further than showing present abnormal conditions that the future may adjust.

"6. That six and seven day weather periods are too ill-defined and irregular to be applicable to the actual work of forecasting.

"7. That advances in the period and accuracy of weather forecasts depend upon a more exact study and understanding of atmospheric pressure over great areas and a determination of the influences, probably solar, that are responsible for normal and abnormal distributions of atmospheric pressure over the earth's surface.

"8. That meteorologists are not antagonistic to honest, well-directed efforts to solve the problem of long-range forecasting; that, on the contrary, they encourage all work in this field and condemn only those who, for notoriety or profit, or through misdirected zeal and unwarranted assumption, bring the science of meteorology into disrepute.

"9. That meteorologists appreciate the importance to the world at large of advances in the period of forecasting and are inclined to believe that the twentieth century will mark the beginning of another period in meteorological science."

THE WEATHER AT CLEVELAND, OHIO. WHAT IT HAS BEEN FOR FIFTY YEARS. SUMMARY OF METEOROLOGICAL OBSERVATIONS. TEMPERATURE, RAINFALL, SNOWFALL, SKY AND WIND.

By G. A. Hyde, Volunteer Observer for United States Weather Bureau.

The years, with us, roll on apace,
  And we make record of the race;
Of Wind and Sky; of Rain and Snow;
  Of Heat and Cold, which all would know.

Fifty years of continued and systematic observations on the weather at Cleveland are presented in this paper, showing interesting characteristics as follows:

Temperature. — High and low, weekly, monthly and yearly averages. Yearly average in decades, showing that there are no uniform deviations from normal. Beginning of the summer and winter in decades, showing that there is no gradual change in seasons, as many suppose. The time of the beginning and ending of the seasons in this locality, which has never been determined before.

Rainfall. — Quantities per hr., day, week, month and year, and periods of no rains.

Snowfall. — The exceptional fall for one day; the amounts per month and year, and the marked difference in the totals for the months and seasons.

Sky. — The months and seasons having the greatest and least cloudiness.

Winds. — The prevailing direction at 7 A.M., 2 P.M. and 9 P.M., for month, season and year.

Observations from which this summary for 50 years was prepared were commenced May 1, 1855.

The hours of observation were 7 A.M., 2 P.M. and 9 P.M., sun-time.

Temperatures were taken with fahr. thermometer.

A part of the maximum and minimum temperatures were furnished through the courtesy of the United States Weather Bureau, of this city.

Temperatures.

The highest observed was 99 degrees, August 12, 1881.

The highest average for one day was 87.8 degrees, July 17, 1887, and the next, 87.5, July 4, 1897.

The highest average for two consecutive days was 85.9 degrees, beginning July 16, 1887.

The highest average for three consecutive days was 84. degrees, beginning July 15, 1887.

The highest average for four consecutive days was 83.6 degrees, beginning June 24, 1858.

The highest average for five consecutive days was 83.3 degrees, beginning June 23, 1858.

The highest average for six consecutive days was 82.6 degrees, beginning June 22, 1858.

The highest average for seven consecutive days was 80.8 degrees, beginning June 22, 1858.

The highest average for fourteen consecutive days was 80.1 degrees, beginning June 20, 1858.

The highest average for one month was 76.7 degrees, beginning July 1, 1868, and the next, 76.4 degrees, June 3, 1891.

The highest average for two months was 74.7 degrees, beginning June 16, 1858.

The highest average for three months was 72.7 degrees, beginning June 13, 1868.

The highest average for one year was 51.2 degrees, in 1894.

The lowest observed was 20 degrees below zero, January 29, 1873.

The lowest average for one day was 10 degrees below zero, January 29, 1873.

The lowest average for two consecutive days was 9.8 degrees below zero, beginning February 9, 1899.

The lowest average for three consecutive days was 7.8 degrees below zero, beginning February 9, 1899.

The lowest average for four consecutive days was 6.5 degrees below zero, beginning February 9, 1899.

The lowest average for five consecutive days was 5.3 degrees below zero, beginning February 9, 1899.

The lowest average for six consecutive days was 4.2 degrees below zero, beginning February 8, 1899.

The lowest average for seven consecutive days was 1.9 degrees below zero, beginning February 7, 1899.

The lowest average for fourteen consecutive days was 7.3 degrees above zero, beginning February 4, 1875.

The lowest average for one month was 13.8 degrees above zero, beginning December 30, 1856.

The lowest average for two months was 15.9 degrees above zero, beginning December 24, 1855.

The lowest average for three months was 18.9 degrees above zero, beginning December 23, 1855.

The lowest average for one year was 45.7 degrees in 1875.

There was an unusually hot spell of weather in June, 1858, the detail of which is as follows:

	7 A.M. deg.	2 P.M. deg.	9 P.M. deg.	Mean. deg.
June 22	77	84	75	79.0
,, 23	82	87	78	81.3
,, 24	83	88	77	81.3
,, 25	84	89	77	81.8
,, 26	81	90	80	82.5
,, 27	84	91	79	83.2
,, 28	80	84	72	77.0
			Mean for 7 days,	80.8

There was an unusually cold spell of weather in February, 1899, the detail of which is as follows:

	7 A.M. deg.	2 P.M. deg.	9 P.M. deg.	Mean. deg.
February 8	6	8	0–5	1.0
,, 9	0–14	0–5	0–10	0–9.8
,, 10	0–18	0–5	0–8	0–9.8
,, 11	0–7	0	0–4	0–3.7
,, 12	0–5	0	0–3	0–2.7
,, 13	0–6	2	1	0–0.5

Weekly Average Temperatures.

Week of Year.		Maximum.		Minimum.		50 Years.
No.	Ending.	Temp.	Year.	Temp.	Year.	Temp.
		deg.		deg.		deg.
1	January 7	43.9	1890	9.6	1884	27.0
2	" 14	42.2	1890	9.7	1893	26.2
3	" 21	41.1	1894	9.3	1857	27.7
4	" 28	44.6	1864	7.8	1897	26.5
5	February 4	44.0	1890	11.6	1905	25.1
6	" 11	40.0	1898	2.8	1895	27.1
7	" 18	43.2	1867	7.0	1875	27.7
8	" 25	39.8	1857	12.5	1885	29.3
9	March 4	50.4	1861	17.0	1884	31.3
10	" 11	53.3	1878	16.9	1856	33.7
11	" 18	49.5	1868	21.0	1894	34.4
12	" 25	46.6	1865	15.5	1885	36.1
13	April 1	46.3	1875	24.5	1856	38.7
14	" 8	55.9	1892	28.2	1881	41.7
15	" 15	56.2	1869	33.4	1885	45.7
16	" 22	64.3	1866	28.4	1875	48.5
17	" 29	61.1	1886	37.3	1874	50.7
18	May 6	68.6	1895	42.9	1890	53.3
10	" 13	67.6	1889	48.2	1885	57.0
20	" 20	70.2	1877	48.6	1888	57.5
21	" 27	71.1	1880	52.8	1883	61.3
22	June 3	74.6	1871	52.8	1889	63.3
23	" 10	74.0	1865	57.3	1869	64.9
24	" 17	76.3	1876	60.4	1855	67.4
25	" 24	78.1	1858	62.1	1881	68.9
26	July 1	81.2	1858	64.5	1892	71.0
27	" 8	77.8	1858	64.2	1892	72.5
28	" 15	78.4	1859	65.8	1865	71.7
29	" 22	79.0	1868	64.8	1871	72.0
30	" 29	78.4	1892	65.9	1871	73.4
31	August 5	79.7	1861	66.7	1801	71.3
32	" 12	80.4	1862	63.2	1884	71.4
33	" 19	74.9	1864	63.8	1879	68.8
34	" 26	74.9	1872	59.8	1866	68.5
35	September 2	74.0	1880	59.1	1885	67.1
36	" 9	77.7	1881	56.5	1883	66.7
37	" 16	74.7	1865	58.5	1879	64.7
38	" 23	70.4	1891	52.3	1875	61.3
39	" 30	74.0	1881	51.4	1893	59.4
40	October 7	69.4	1862	47.8	1876	56.8
41	" 14	68.0	1879	42.9	1876	53.7
42	" 21	64.0	1867	42 2	1869	51.4
43	" 28	59.0	1874	35.8	1869	48.4
44	November 4	56.3	1860	39.0	1878	46.8
45	" 11	52.4	1874	32.4	1869	44.0
46	" 18	49.4	1879	28.7	1880	40.5
47	" 25	50.3	1883	16.5	1880	37.0
48	December 2	51.4	1864	24.5	1872	34.4
49	" 9	45.7	1879	22.2	1882	33.2
50	" 16	46.0	1862	19.5	1868	33.5
51	" 23	48.7	1877	15.3	1872	30.2
52	" 31	42.9	1875	12.5	1880	28.9

Monthly — Maximum and Minimum.

	Maximum.			Minimum.		
Month.	Temp.	Day.	Year.	Temp.	Day.	Year.
	deg.			deg.		
January.......	70	22	1874	0–20	29	1873
February......	72	16	1883	0–18	10	1899
March.........	79	29	1905	0– 5	3	1868
April..........	86	30	1872	15	17	1875
,, ...	..	18	1896	..	..	..
,, ...	..	30	1899	..	..	..
May..........	93	30	1879	28	1	1876
June..........	96	28	1874	37	6	1894
July...........	97	7	1874	48	9	1894
August........	99	12	1881	45	31	1890
September.....	98	6	1881	36	26	1887
October.......	88	1	1877	24	25	1889
,, ...	..	..	..	..	26	1887
November.....	74	1	1888	0– 4	26	1857
December.....	68	31	1875	0–14	22	1872

Monthly Averages.

Month.	Maximum.		Minimum.		50 Years. Temp.
	Temp.	Year.	Temp.	Year.	
	deg.		deg.		deg.
January....................	40.1	1880	14.7	1857	26.9
February..................	37.5	1882	17.1	1885	28.0
March.....................	46.6	1898	24.9	1856	34.9
April......................	53.8	1878	37.8	1857	46.8
May.......................	65.8	1880	51.5	1867	58.0
June......................	73.6	1865	62.4	1903	67.7
July.......................	77.7	1868	68.0	1871	71.9
August....................	74.0	1862	65.4	1866	69.9
September................	71.6	1881	59.0	1883	63.3
October...................	59.7	1900	44.9	1869	52.1
November.................	48.4	1902	31.9	1880	40.4
December.................	42.2	1889	21.0	1876	31.2

Yearly Averages.

1st Decade.		2d Decade.		3d Decade.		4th Decade.		5th Decade.	
Year.	Temp.	Year.	Temp.	Year.	Temp.	Year.	Temp.	Year.	Temp.
	deg.		deg.		deg.		deg.		deg.
1855	48.4	1865	50.8	1875	45.7	1885	46.6	1895	48.5
1856	46.3	1866	48.8	1876	48.9	1886	48.7	1896	50.1
1857	47.4	1867	49.1	1877	50.7	1887	49.4	1897	49.5
1858	50.1	1868	47.2	1878	50.5	1888	48.2	1898	50.7
1859	49.8	1869	48.0	1879	49.3	1889	50.0	1899	49.8
1860	50.1	1870	49.5	1880	50.5	1890	50.7	1900	50.6
1861	51.1	1871	49.2	1881	50.7	1891	50.5	1901	49.1
1862	51.0	1872	47.8	1882	49.8	1892	48.9	1902	49.5
1863	51.0	1873	47.9	1883	49.6	1893	49.1	1903	48.9
1864	50.5	1874	49.8	1884	48.9	1894	51.2	1904	47.2
Mean.	49.6	Mean.	48.8	Mean.	49.5	Mean.	49.3	Mean.	49.4

Average temperature for 50 years, 49.26 degrees.

Dividing the year into two seasons, warm and cold, of six months' duration each, the warm season begins April 25, and has an average temperature of 64.5 degrees; and the cold season begins October 24, and has an average temperature of 34.3 degrees.

Dividing the year into four seasons of spring, summer, autumn and winter, calling summer the 91 warmest days and winter the 91 coldest days, summer begins June 11, with a daily mean of 65.7 degrees, and ends September 9, with a daily mean of 66.3 degrees, and has an average temperature of 70.1 degrees. Winter begins December 6, with a daily mean of 32.8 degrees, and ends March 6, with a daily mean of 32.3 degrees, and has an average temperature of 28.6 degrees. Spring, therefore, begins March 7 and ends June 10, a period of 95 days, and has an average temperature of 48.7 degrees, and autumn begins September 10 and ends December 5, a period of 87 days, and has an average temperature of 49.3 degrees.

The highest average for 91 days of summer weather was 72.8 degrees, beginning June 13, 1864, and the lowest average for 91 days of summer weather was 68.0 degrees, beginning June 2, 1883.

Taking the average for 50 years, the days of highest temperature are July 15, 73.5 degrees; July 16, 73.5 degrees, and July 17, 73.4 degrees; and the lowest are February 4, 23.7 degrees; February 2 and 5, 24.4 degrees and January 10, 24.5 degrees.

The earliest beginning of summer was May 20, 1887, and had an average temperature of 72.0 degrees.

The latest beginning of summer was June 26, 1869, and had an average temperature of 68.9 degrees.

The longest summer season that had the average temperature of the 50 summers (70.1 degrees), occurred in 1864, commencing May 16 and ending September 23, a period of 131 days; and the shortest summer season of same average (70.1 degrees) occurred in 1883, commencing July 2 and ending July 29, a period of 28 days.

The highest average for 91 days of winter was 36.0 degrees, beginning January 1, 1890, and the lowest average for 91 days was 18.9 degrees, beginning December 23, 1855.

The earliest beginning of winter of 91 days was November 15, 1858, and had average temperature of 31.1 degrees.

The latest beginning of winter of 91 days was January 10, 1876, and had an average temperature of 32.8 degrees.

The longest winter season having the average temperature of the 50 winters (28.7 degrees), commenced October 22, 1855, and ended April 13, 1856, a period of 175 days; and the shortest winter season of same average commenced January 16, 1890, and ended January 24, a period of 9 days.

The time of beginning of summers and winters in decades and their average temperature are as follows:

Decade.	Summer.		Winter.	
	Month and Day.	Degree.	Month and Day.	Degree.
First	June 18	70.8	December 8	29.1
Second	" 5	70.3	" 8	27.4
Third	" 11	69.8	" 8	29.8
Fourth	" 10	69.5	" 8	29.9
Fifth	" 9	69.5	" 5	27.5
Average	June 11	70.1	December 8	28.6

Rainfall (Including Melted Snow).

The most rapid rainfall was 0.85 in., and fell in 10 min., June 3, 1891, being at the rate of 5.10 in. per hr.

The greatest fall within 1 hr. occurred August 15, 1885, being 1.53 in. in 51 min., the most of which fell in 40 min.

The greatest fall in 10 hr. was 3.13 in., September 12 and 13, 1878.

The greatest fall in 1 day, or 24 hr., was 4.67 in., September 12 and 13, 1878.

The greatest fall in 1 week was 6.49 in., September 10 to 13 inclusive, 1878.

The greatest fall in 1 month was 10.33 in., in June, 1855.

The least fall in 1 month was 0.25 in., in February, 1877.

The greatest fall in 1 year was 49.66 in., in 1878.

The least fall in 1 year was 25.28 in., in 1856.

The average yearly rainfall for 50 years is 38.04 in.

The longest period in which less than 0.01 in. of rain fell was 23 days, May 30 and June 21 inclusive, 1864.

The least rainfall in 34 days was 0.13 in., May 26 and June 28 inclusive, 1864.

The least rainfall in 42 days was 0.42 in., July 31 and September 10 inclusive, 1881.

The least rainfall in 49 days was 0.52 in., July 24 and September 10 inclusive, 1881.

The least rainfall in 55 days was 0.79 in., July 23 and September 15 inclusive, 1881.

Weekly Rainfall (Including Melted Snow).

Week of Year.		Greatest Fall.		Least Fall.		Average. 50 Years.
No.	Ending.	Inches.	Year.	Inches.	Year.	Inches.
1	January 7	2.64	1874	0.02	1866	0.61
2	,, 14	1.06	1870	0.01	1894	0.57
3	,, 21	1.84	1904	0.02	1856	0.65
4	,, 28	1.77	1876	0.00	1892	0.54
5	February 4	1.99	1878	0.03	1866, 1882	0.54
6	,, 11	4.54	1887	0.02	1877	0.72
7	,, 18	1.80	1891	0.00	1875	0.64
8	,, 25	2.39	1874	0.08	1864	0.61
9	March 4	1.99	1875	0.00	1886, 1894	0.62
10	,, 11	1.84	1868	0.02	1858	0.62
11	,, 18	1.94	1878	0.00	1889	0.54
12	,, 25	2.74	1898	0.00	1902	0.69
13	April 1	2.64	1904	0.00	1893, 1897	0.82
14	,, 8	2.09	1873	0.01	1877	0.60
15	,, 15	2.80	1859	0.00	1877	0.70
16	,, 22	2.55	1870	0.00	1886, 1890	0.63
17	,, 29	2.08	1880	0.00	1888, 1891, 1892	0.57
18	May 6	4.29	1892	0.00	1859	0.90
19	,, 13	3.04	1901	0.00	1871, 1879, 1896, 1903	0.69
20	,, 20	5.58	1893	0.00	1885, 1900, 1903	0.83
21	,, 27	3.26	1883	0.00	1857, 1881	0.77
22	June 3	2.89	1891	0.00	1864, 1873, 1884, 1895	1.07
23	,, 10	5.43	1881	0.00	1862, 1864, 1867	0.99
24	,, 17	5.14	1857	e.00	1860, 1864, 1884	0.75
25	,, 24	5.15	1866	0.00	1858, 1861, 1865, 1872, 1880, 1904	0.80
26	July 1	4.38	1902	0.00	1871, 1884, 1886, 1887, 1901	0.89
27	, 8	3.39	1904	0.00	1855, 1868, 1886, 1888	0.97
28	,, 15	3.23	1880	0.00	1859, 1866, 1868, 1898	0.80
29	,, 22	4.71	1872	0.00	1856, 1864, 1868, 1878, 1884	0.84

WEEKLY RAINFALL (INCLUDING MELTED SNOW).

Week of Year.		Greatest Fall.		Least Fall.		Average. 50 Years.
No.	Ending	Inches.	Year.	Inches.	Year.	Inches.
30	July 29	3.29	1870	0.00	1859, 1866, 1871, 1887	0.82
31	August 5	3.15	1875	0.00	1870, 1874, 1877, 1901	0.73
32	" 12	2.43	1855	0.00	1864, 1870, 1890, 1894	0.52
33	" 19	2.08	1901	0.00	1865, 1874, 1884, 1899, 1903	0.79
34	" 26	2.92	1871	0.00	1855, 1856, 1873, 1881, 1888, 1893, 1894	0.62
35	September 2	4.84	1901	0.00	1856, 1874, 1875, 1876, 1889, 1894, 1896, 1899	0.95
36	" 9	3.47	1870	0.00	1871, 1886, 1901	0.86
37	" 16	6.49	1878	0.00	1857, 1870, 1900	0.91
38	" 23	3.14	1876	0.00	1858, 1867	0.73
39	" 30	2.66	1884	0.00	1867, 1885, 1891, 1894, 1897	0.85
40	October 7	2.67	1903	0.00	1866, 1882, 1899	0.88
41	" 14	3.25	1893	0.00	1865, 1871	0.73
42	" 21	2.57	1877	0.00	1874, 1889, 1894	0.60
43	" 28	2.91	1878	0.00	1901	0.68
44	November 4	2.73	1861	0.00	1887, 1904	0.61
45	" 11	3.48	1857	0.00	1865, 1883	0.87
46	" 18	2.43	1879	0.00	1865	0.85
47	" 25	2.22	1895	0.02	1882	0.65
48	December 2	2.04	1869	0.01	1899	0.71
49	" 9	2.28	1878	0.05	1881	0.64
50	" 16	3.03	1873	0.01	1871	0.69
51	" 23	1.93	1881	0.00	1894	0.59
52	" 31	1.96	1862	0.06	1860	0.64

MONTHLY RAINFALL (INCLUDING MELTED SNOW).

Month.	Greatest Fall.		Least Fall.		Average 50 Years.
	Inches.	Year.	Inches.	Year.	Inches.
January	6.15	1870	0.99	1872	2.52
February	7.54	1887	0.25	1877	2.61
March	5.89	1877	0.62	1856	2.94
April	5.80	1860	0.98	1863	2.72
May	8.72	1892	0.75	1877	3.74
June	10.33	1855	0.34	1864	3.97
July	10.15	1870	0.45	1868	3.76
August	7.83	1861	0.28	1881	3.09
September	9.23	1878	1.03	1871	3.77
October	6.74	1890	0.74	1886	2.94
November	5.39	1857	0.41	1904	3.22
December	5.09	1873	1.24	1900	2.77
Year	49.66	1878	25.28	1856	38.04

SNOWFALL.

The most remarkable snowstorm noted in the 50 years, occurred January 31, 1878, between the hours of 4 A.M. and 9 P.M., in which time there fell 22 in. of snow, 18 in. of which fell between 8 A.M. and 4 P.M., 8 hr., and 4 in. fell in 20 min. This snow melted produced 1.98 in. of water.

Of ordinary snow, about 10 in. will make 1 in. of water.

December 16, 1882, there fell 2.5 in. of snow, which made 0.07 in. of water, or at the rate, 37 in. of snow to make 1 in. of water.

Of the first snowfall of the season, the earliest occurred September 30, 1888, and the latest, November 29, 1865.

Of the last snowfall of the season, the latest occurred May 27, 1902, and the earliest, March 24, 1878.

The greatest total snowfall for the winter season occurred during the winter season of 1880–81, showing depths for months as follows:

November, 1880	14.4 in.
December, 1880	14.6 ,,
January, 1881	20.9 ,,
February, 1881	24.4 ,,
March, 1881	23.2 ,,
April, 1881	4.3 ,,
Total for the season	101.8 ,,

The least total snowfall for the winter season occurred during the winter season of 1865–66, showing depths for months as follows:

November, 1865	0.1 in.
December, 1865	2.2 „
January, 1866	6.1 „
February, 1866	4.9 „
March, 1866	9.2 „
April, 1866	0.1 „
Total for the season	22.6 „

The snowfall per month has been as follows:

Month.	Greatest.		Least.		Average. 50 Years.
	Inches.	Year.	Inches.	Year.	
September	0.2	1888	None.	50 years.	Trace.
October	5.6	1878	None.	31 years.	0.6
November	18.3	1874	None.	1855	6.7
December	36.4	1886	0.3	1889	10.7
January	31.7	1878	1.7	1858	12.1
February	24.4	1881	0.5	1877	10.7
March	24.0	1877	0.5	1902	8.8
April	16.8	1857	None.	8 years.	3.5
May	1.4	1875	None.	44 years.	0.1
Season					53.2

Cloudiness of the Sky.

In the following statement the varied average cloudiness of the sky is illustrated by a ratio, in which 0.0 stands for clear sky, and 10.0 for the sky entirely obscured by clouds.

For Month, Season and Year.

Month.	Maximum.		Minimum.		Average. Ratio 50 Years.
	Ratio.	Year.	Ratio.	Year.	
January	8.8	1871, 1891	6.2	1857, 1864, 1885	7.6
February	8.5	1884	5.4	1877	7.3
March	8.5	1899	5.0	1860	6.8
April	7.3	1873, 1894	4.3	1877	6.0
May	7.3	1892	3.7	1895	5.4
June	7.1	1869	3.2	1895	4.9
July	6.7	1869	3.3	1890, 1892, 1894, 1897, 1898	4.5
August	6.1	1903	3.2	1889	4.5
September	7.5	1876	2.7	1897	5.0
October	7.6	1881, 1890	4.1	1871	6.0
November	9.4	1858	6.1	1893	7.6
December	9.3	1879	5.5	1891	7.9
Spring	7.1	1904	5.3	1877	6.1
Summer	6.3	1869	3.6	1893	4.6
Autumn	7.6	1876	5.1	1893	6.2
Winter	8.4	1897–8	6.4	1891–2	7.6
Year	6.9	1869	5.4	1895	6.1

In the following table is illustrated the average cloudiness of the sky at 7 A.M., 2 P.M. and 9 P.M. during the different months of the year and for the year.

Month.	7 A.M.	2 P.M.	9 P.M.
January	8.1	7.7	7.1
February	7.7	7.4	6.7
March	7.2	6.9	6.4
April	6.3	6.1	5.4
May	5.6	5.6	4.6
June	5.5	5.1	4.2
July	4.9	4.9	3.9
August	4.9	4.8	3.6
September	5.5	5.4	4.3
October	6.4	6.3	5.6
November	7.9	7.7	7.0
December	8.5	8.0	7.4
Year	6.6	6.4	5.5

WINDS.

Table of proportion of wind directions represented in hundredths. Average for 50 years:

DIRECTION OF THE WIND.

MONTH, SEASON, YEAR.	7 A.M.								2 P.M.								9 P.M.								AVERAGE.							
	N.	NE.	E.	SE.	S.	SW.	W.	NW.	N.	NE.	E.	SE.	S.	SW.	W.	NW.	N.	NE.	E.	SE.	S.	SW.	W.	NW.	N.	NE.	E.	SE.	S.	SW.	W.	NW.
March	9	10	4	13	24	12	14	14	15	13	2	7	13	9	21	20	10	13	6	12	22	7	17	13	11	12	4	11	20	9	17	16
April	11	11	4	15	26	10	12	11	19	22	1	6	11	6	16	19	11	17	5	12	27	7	14	7	14	17	3	11	21	8	14	12
May	13	11	2	12	31	13	9	9	25	21	1	4	11	7	13	18	12	16	5	12	33	7	10	5	17	16	3	9	25	9	10	11
Spring	11	11	3	13	27	12	12	11	20	19	1	6	12	7	17	19	11	15	5	12	28	7	14	8	14	13	4	11	22	9	14	13
June	10	8	3	15	31	15	12	6	27	17	1	3	10	9	15	18	9	11	4	14	38	10	10	4	16	12	3	11	26	11	12	9
July	9	9	3	12	37	16	9	5	32	15	1	2	8	7	13	22	10	9	4	12	46	7	7	5	17	11	3	9	30	10	10	10
August	8	7	4	17	41	12	6	5	32	18	1	3	9	6	9	22	8	10	4	15	45	6	7	5	16	12	3	12	32	8	7	10
Summer	9	8	3	15	37	14	9	5	30	17	1	3	9	7	12	21	11	11	4	12	39	9	8	6	17	12	3	10	28	10	10	10
September	6	6	4	22	41	10	4	7	23	17	2	5	16	8	12	17	8	8	5	15	39	9	9	7	13	11	4	11	32	9	9	11
October	8	5	5	16	38	14	5	9	15	12	2	7	20	13	15	16	8	7	2	12	40	11	11	9	10	8	3	12	33	13	10	11
November	6	4	3	12	35	19	9	12	9	7	3	7	24	19	17	14	6	4	4	10	34	16	16	10	7	5	3	10	31	18	14	12
Autumn	7	5	4	17	38	14	6	9	16	12	2	6	20	13	15	16	7	6	4	12	38	12	12	9	10	8	3	11	33	13	11	11
December	4	5	5	13	30	22	12	9	6	3	3	10	22	20	22	11	6	5	4	10	29	17	21	8	5	5	4	11	27	20	19	9
January	6	6	4	11	30	19	15	9	7	7	3	8	23	17	23	12	7	7	4	9	27	17	21	8	7	7	4	9	26	17	20	10
February	6	7	4	13	27	15	17	11	11	9	2	8	20	13	23	14	7	9	4	10	29	11	19	11	8	8	3	10	26	13	20	12
Winter	6	6	3	12	29	19	15	10	8	7	3	9	22	17	22	12	7	7	4	10	28	15	20	9	7	7	3	10	26	17	20	10
Year	8	8	3	14	33	15	10	9	18	14	2	6	16	11	16	17	9	10	4	11	33	8	13	8	12	10	3	10	27	12	14	11

WINDS.

Table showing the greatest average proportion of wind directions, in hundredths, contained in any quadrant in 50 years.

MONTH AND SEASON.	7 A.M.				2 P.M.				9 P.M.				AVERAGE.			
March	S.	SW.	W.	49	W.	NW.	N.	56	S.	SW.	W.	47	S.	SW.	W.	46
April	SE.	S.	SW.	51	NW.	N.	NE.	60	SE.	S.	SW.	47	NW.	N.	NE.	42
May	SE.	S.	SW.	55	NW.	N.	NE.	64	SE.	S	SW.	52	NW.	N.	NE.	44
Spring	SE.	S.	SW.	50	NW.	N.	NE.	58	S.	SW.	W.	48	S.	SW.	W.	44
June	SE.	S.	SW.	62	NW.	N.	NE.	62	SE.	S.	SW.	62	S.	SW.	W.	49
July	SE.	S.	SW.	65	NW.	N.	NE.	69	SE.	S.	SW.	65	S.	SW.	W.	50
August	SE.	S.	SW.	70	NW.	N.	NE.	72	SE.	S.	SW.	66	SE.	S.	SW.	49
Summer	SE.	S.	SW.	66	NW.	N.	NE.	68	SE.	S.	SW.	60	SE.	S.	SW.	49
September	SE.	S.	SW.	73	NW.	N.	NE.	57	SE.	S.	SW.	63	SE.	S.	SW.	52
					S.	SW.	W.	48								
October	SE.	S.	SW.	68	W.	NW.	N.	46	SE.	S.	SW.	63	SE.	S.	SW.	58
November	SE.	S.	SW.	66	S.	SW.	W.	60	S.	SW.	W.	66	S.	SW.	W.	63
Autumn	SE.	S.	SW.	69	S.	SW.	W.	48	SE.	S.	SW.	62	SE.	S.	SW.	57
December	SE.	S.	SW.	65	S.	SW.	W.	64	S.	SW.	W.	67	S.	SW.	W.	67
January	S.	SW.	W.	64	S.	SW.	W.	63	S.	SW.	W.	65	S.	SW.	W.	63
February	S.	SW.	W.	59	S.	SW.	W.	56	S.	SW.	W.	59	S.	SW.	W.	59
Winter	S.	SW.	W.	63	S.	SW.	W.	61	S.	SW.	W.	63	S.	SW.	W.	63
Year	SE.	S.	SW.	62	W.	NW.	N.	51	S.	SW.	W.	58	S.	SW.	W.	54

What gain we by this flood of facts?
Can we forecast God's wondrous acts?
No! what has passed we'll see no more,
But wonder what is still in store.

The foregoing is meteorological history. History never repeats itself, and the grouping of forces which produced the weather during the past fifty years will never be so repeated. The statistics contained in the foregoing paper are interesting in that they summarize a fifty years' record of varied atmospheric conditions, but they reveal no true cycles which will enable us to make predictions for the future.

Of course, the seasons will come and go as they always have, and exhibit the peculiar conditions that belong to each of them, but the deviation from normal will likely be as astonishing in the future as in the past, and as much at variance with the seasonable as wonderful in extent.

BREAKAGE IN SEWER CONDUITS: ITS CAUSE, EFFECT AND PREVENTION.

By Alexander Potter, C. E.

[Read before the Sanitary Section of the Boston Society of Civil Engineers, October 4, 1905.]

Data in relation to the extent of broken pipe in sewer trenches are difficult to secure. There are a number of reasons for this. Perhaps the first and most general is ignorance of its existence. Many who wonder why certain lines of pipe leak and cause trouble, even where close personal attention was given to the construction work by the engineer in the hope of securing satisfactory work, would cease to wonder if some of these ditches were opened up and the condition of the pipes exposed.

Again, it is pretty well understood that, barring the admission of ground water, the existence of cracked pipe in a sewer does not affect its use as a water carrier except in quicksand excavation. When fine running sand is encountered, the necessity for tight joints is essential, for it is often cheaper to relay long stretches of pipe than to remove the sand in the pipe after it enters the sewer and packs itself therein. In the larger sizes this admission of quicksand will tend to remove the support from the pipe and cause rupture.

Generally, it is only when the pipe is so badly broken as to require replacement, that information is sought and obtained, — and usually meager information at that, especially as to the cause of breakage. An engineer is naturally not eager to make known the existence of cracked pipe unless it is liable to cause trouble. As we have little or no published data relating to the extent of broken sewer pipe in constructed sewerage systems, and there is so little known about it, engineers have gone on building pipe sewers under specifications which will in the opinion of the writer produce broken pipe in all the larger sizes.

It is safe to say that cracked and broken pipe is general to an extent much greater than people think. More broken pipes exist to-day in sanitary sewers laid with the utmost precaution and with well selected pipe than exist in ordinary drains at greater depth laid with inferior pipe.

The use of cement and vitrified pipe on sanitary sewer work is of comparatively recent origin, and much of the knowledge secured concerning its behavior was based on its use in culverts and drains, in which case the admission of water through the joints was a positive advantage in lowering the ground-water plane, and consequently little attention was given to the joints. The omission or imperfect application of the cementing medium did not impair the strength of the drain. Each pipe assumed its own bearing, practically independent of its neighbor. When, however, care is taken to properly joint the pipe, as is the case in the laying of a sanitary sewerage system, an element of weakness is introduced, due to the inelasticity of a properly jointed line of pipe, and the imperfect nature of the foundation of one pipe cannot be lightly considered, as it may, and frequently does, vitally affect the stability of many of its neighbors.

In laying pipe through dry sand, loam or clay free from water, no difficulty should be experienced in securing a good foundation, for the bottom of the ditch can usually be cut out to the approximate shape of the pipe, and a natural foundation is thus afforded. In rock cuts and wet ditches, however, this is impossible, and the natural support of the pipe must be replaced by an artificial one, and the problem confronting the engineer is a most serious one.

It is a safe assumption, and one which is amply borne out in practice, that the presence of ground water in pipe sewer trenches at least doubles the actual cost of sewer construction if the best work is to be secured. In the endeavor to prevent this increase in cost to his clients, the engineer is tempted to

take chances. Indeed, this risk, admitted or not, is taken by engineers all over the country, with the result that to-day there is much defective work buried, like the "doctor's mistakes," under the ground.

A common form of foundation in wet trenches consists of well rammed gravel, broken stone or cinders, usually with the addition of underdrains. Concrete is also used quite generally. A proper foundation is also secured in yielding material by using timber under the pipe, either in the form of planking, some form of cradle, blocking or piling. The ordinary and usual methods of replacing backfilling material around the pipe are not sufficiently thorough to insure the larger sizes of pipe from cracking or collapse. The statement that it is practically impossible to properly backfill with the material taken from the trench in general, under and around the pipe on the larger sizes where the natural foundation of the pipe has been removed, will not be seriously challenged. Ruptured pipe can only be properly avoided by filling the space under the pipe and up to one third of its height with concrete, which can be very lean, if the pipes are properly jointed, or by filling up to the top of the pipe with fine sand, flooded to enable the sand to flow entirely under and around the pipe, thus insuring a hard foundation. The use of the sand in this way is only practicable where some jointing material which immediately hardens is used.

It is a fact that most specifications have excellently worded clauses about backfilling which are impractical in many ways. In many trenches it is absolutely impossible with any tool to ram the backfilling material below and around the lower portion of the pipe. Again, it is difficult to get the workmen to appreciate the importance of thoroughly treating each and every foot of pipe along the line of the sewer with the same faithful care, and it is often impossible for the inspector to watch both pipe laying and backfilling, even when he is inclined to be most watchful. The material at hand for backfilling around the pipe, even when carefully selected, may not be suitable. If these objectionable conditions are not well guarded against, the resultant breaking of one pipe is liable to rupture a hundred.

Another element of uncertainty in sewer-pipe laying is the character of the pipe itself. The writer has often secured better results in standards of vitrified pipe from one manufacturer than from double-strength pipe of another maker, and material from the same factory often varies within wide limits. Whether the pipe be shale, fire-clay, or "stoneware," so called, does not

render it immune from breakage under certain conditions. This lack of uniformity in output is equally noticeable in molded cement pipe.

In the construction of the Joint Trunk Sewer in New Jersey, recently completed, 150 miles of sanitary sewers, trunks and laterals in the municipalities interested, exceptionable facilities were afforded the writer to study this question, for over 65 per cent. of all the ditches were dug through drift saturated with water, requiring the disturbance of the natural bed for the pipe.

In constructing a separate system of sewers, the aim of the engineer is to secure water-tight work, but in water-bearing ground nothing is more difficult of attainment. The Joint Sewer work afforded an exceptionable opportunity to determine what could be done in wet work. It was observed that if the joints in sewer pipes were properly made, a cylinder of indefinite length was produced, and very slight irregularities in the grade or slight settlements in the foundations greatly reduced the power of the pipe to resist cracking, as the pipe is practically inelastic.

In experiments made by Parkin and described in the Transactions of the Liverpool, England, Engineering Society, Vol. XVII, 15-in. pipe, supported on permanent pillars 8 ft. 4 in. apart, broke under a gradually applied load of 2 491 lb., including the weight of the pipe, and showed a maximum deflection of 1-30 of an inch at the center. If the trench is backfilled before the jointing material hardens, the sewer will better adjust itself to the irregularities of grade or to settlement in the foundation. This will reduce the liability to rupture, but will increase the percentage of leakage through injury to the joints. The injury to the joints is caused by the raising of the water in the trench, softening the cement and causing it to sag or drop off altogether; or, if the cement has had an initial set before the water is allowed to rise, the pressure of the earth ruptures the joints through settlement, or the cement is broken off by careless backfilling. As a matter of fact the expression, "careless backfilling," is hardly applicable, for to secure even reasonably satisfactory work the highest care must be given to backfilling around the pipe, — greater care, in fact, than can reasonably be expected from the class of men employed upon this sort of work.

From examination of constructed lines of pipe sewers it is almost certain that if a pipe line ruptures at all it will do so at the time of the first heavy rainstorm after the trench has been

completely backfilled, provided the frost is out of the ground when the rain occurs.

In the work under discussion 3-ft. lengths of pipe were used throughout, and to increase the possibility of securing tight joints, the writer adopted the plan of laying many of the sewers in wet trenches with yielding foundations on cradles. The cradle or wooden underdrain, as it might be called, consists of 4 pieces of 2 in. by 4 in. timber laid flat, 6 in. centers, and 9 ft. or 12 ft. long. On these were nailed 4 in. by 4 in. 1.5-ft. centers. The pipe, held by wedges nailed in position, rested on the 4 in. by 4 in. timbers. The space below the pipe acted as a channel to drain the water in the trench to a sump, and prevented its rising on the joints. This construction is cheaper than the ordinary form of broken stone or gravel foundation with tile underdrain. A questionable feature of this construction, however, is that it leaves a greater area under the pipe to be backfilled, and in clay or other compressible soils this is objectionable. Imperfect backfilling under and around the pipe is the principal cause of failure in sewer pipe.

On the construction of the work referred to, on the larger sizes, all the pipe laid one day was inspected before starting work the next day, and, if found to be satisfactory, passed upon. When the work was about half completed, the writer made a personal examination, and himself crawled bodily through several sections of the 24-in. sewers in deep cuts and other parts where there was a possibility of breakage, and, finding some broken pipe, notified the contractor. The contractor, at a loss to explain its existence, attributed the breaks, which were not many, to the use of the timber supports, and requested permission to use the old form of tile underdrain and broken stone or gravel foundation on the work from that time. This permission was granted. The contractor, desirous of throwing the responsibility of the former breaks upon the method of pipe laying previously used, took more than ordinary care with the work as it then proceeded, including the jointing of the pipe with cement. The joints were in consequence practically perfect, but the ditch was not refilled until the joints had had twenty-four hours' set. The gravel was then thoroughly rammed under and well up around the pipes. In fact no more perfect work could have been performed. The depth of the trench at this point ran from 6 to 8 ft., and standard thickness of pipe was being used. When 200 ft. of this work had been laid and the backfilling completed, an examination of it was made and it was

found to be broken almost from end to end, and actually collapsed at two points. Had the ditch been refilled before the cement hardened the liability to rupture would have been lessened.

A systematic inspection of all completed sections was then started and all cracked pipe either replaced or surrounded with concrete. On all sewers 20 in. in diameter and over, an assistant engineer crawled through every length of pipe laid. On sewers of smaller diameter, an examination was made with the aid of mirrors. In some detail the record of breaks was as follows:

On 26 303 ft. of 24-in. pipe, most of which was standard thickness, 24 different breaks occurred on 24-in. pipe, aggregating 1 500 ft., each break running from 15 ft. to 150 ft. Nearly all of the breaks occurred either in gravel or rock cuttings. No breaks were found in quicksand. The proportion of breakage on gravel or broken stone foundation with underdrain to the total length of sewer laid with this class of foundation was much greater than the proportion of breakage on timber foundation to the total length of sewer laid on this latter class of foundation. The depths of cuts where breaks occurred varied from 6 ft. to 20 ft., but more broken pipes were found at the lesser depths.

On 8 197 ft. of 22-in. standard pipe, at depths varying from 6 ft. to 20 ft., much of which was laid on timber foundation and none on gravel or broken stone, not a single cracked pipe was found.

Fourteen breaks occurred on 4 382 ft. of 20-in. pipe, aggregating 500 ft. in all. All of these breaks occurred in rock cuttings where the pipe was temporarily supported on blocks until selected material was rammed solidly around and under the pipe. A close inspection of the uncovering of the pipe revealed the fact that at certain places sufficient space had not been left at the springing line of the pipe to allow room for the proper ramming of the backfilling around the lower half of the pipe.

On the short stretch of 18-in. pipe laid on a heavy grade, the sewer collapsed shortly after construction, due to the flood of water washing out the newly filled-in material over the sewer under the macadam pavement, which dropped on the pipe and ruptured it.

Concreting under all pipe in wet trenches on this work would have added $125 000 to the cost of the sewers, which was $1 680 000 for 145 miles of sewers. The total cost of replacing the broken pipe was less than $5 000. This did not stop all leakage, but the actual amount of leakage on the entire system

was less than 15 per cent. of the capacity. Had concrete been generally used and the same care given in jointing the pipe, the leakage could have been reduced fully 50 per cent. The value of the additional capacity thus secured for sewage purposes would have been $148 170. In designing the sewer, provision was made for ground water infiltration of 33 per cent. of the capacity. This is not an unusual allowance.

When laboratory experiments show in some cases a leakage in a 6-in. pipe, laid with ordinary care, at the rate of as much as 50 000 gal. per mile per day, is it any wonder, with the additional liability of leakage through cracked pipe, that at least one case is recorded where the leakage in a system with a 24-in. main sewer is only limited by the size of the sewer itself? The writer refers to the "town in New York State," not named, given in a list of towns where records of leakage were obtainable, in an article read by Prof. A. P. Folwell before the American Society of Municipal Improvement a few years ago.

A number of observations in pipe sewers laid under similar conditions have been made to determine the relation between the rate of leakage and the size of the sewer, and while such a relation is hard to establish, the writer's observations so far warrant the conclusion that in general in pipe sewers it varies directly as the square of the diameter of the sewer. This is an important consideration in the design of a sewerage system where large provision is made for ground-water infiltration, for in thus increasing the size of the conduit we introduce a factor that calls for a still greater increase in size.

The writer is free to confess that he is not able to account in each case for the cause of the breakage. For instance, on one stretch of 2 000 ft. of 24-in. pipe, not a single piece of cracked pipe was discovered, while next to it, under exactly the same conditions, but laid under another inspector, 800 ft. of pipe was broken nearly its entire length. I have no reason to think that one inspector was less efficient than the other. At the last mentioned point, in part consideration of the right of way, we were obliged to replace an open ditch about 300 ft. long with a 24-in. drain. For this work rejected pipe from the sewer construction was used almost exclusively, omitting cement in the joints. This drain has been down two years and has not developed a single crack.

In the experience of the writer in replacing sewer lines which have outgrown their capacity, more unexplainable breakage has been discovered on cement sewers than on vitrified pipe sewers.

Much information about broken pipe in systems throughout the country has come to the writer through reliable sources, the truth of which there is no reason to doubt, but which I think it unnecessary and hardly proper to refer to specifically, as it is not my own work, It is to be hoped, however, that other engineers will be frank enough to contribute of their experience, so that as far as possible facts may be obtained and the proper use of sewer pipe standardized.

If the writer is credibly informed, a large city in the middle West, favorably situated for the use of sewer pipe, has recently abandoned its use on all sizes over 18 in. because of the condition in which some lines of pipe already laid have been found.

It is vitally important to engineering that such a case should be recorded and the facts given in detail.

To what extent, if any, breakage is attributable to the use of 3-ft. lengths might form a basis of inquiry. A reduction of 33 per cent. of the number of joints is always a sufficient warrant for the use of 3-ft. over 2-ft. lengths of pipe, if there is no other offset save the small increase in cost. As 80 per cent. of the breakage on the work referred to occurred in ditches less than 10-ft. deep, the use of standard pipe on much of this work cannot account for the failure of the pipe.

The writer is of the opinion that the larger sizes of vitrified pipe should not be used in sanitary sewer construction, say on 20-in. and over, except upon a concrete base, and the relative cost of other material should be compared upon this assumption.

On sizes smaller than 20-in., concrete should be used under the vitrified pipe far more generally than it is at present, and if the contractor knows that the use of concrete is to be general throughout the work, the increase in his price will not be excessive.

Engineers engaged in sewer construction have assumed risks in the past for the sake of keeping down the costs on sewer construction which oftentimes, at best, on account of the great uncertainties in connection with this class of work, exceed their estimates. When broken pipe results they are not relieved from criticism even when the repairs can be effected on a tithe of what it would cost to adopt a method of construction that would eliminate all chance of breakage or when the breakage is the result of careless inspection over which the engineer may or may not have full control.

The writer trusts that the facts set forth and the opinions advanced in this paper, while not all new, will call forth a general discussion and criticism from both engineers and contractors.

Unquestionably much-needed and valuable knowledge upon this subject may thus be secured.

DISCUSSION.

Mr. George Bowers. — I should like to ask the speaker if the cracks are generally in the same part of the pipe, and in what place in the pipes the cracks generally occur.

Mr. Potter. — The cracks occurred usually on 4 quarters of the pipe, with a preponderance of top and bottom cracks. The end of the cracks on one pipe would be coincident with the beginning of the cracks in the next pipe. This would be true for 4 or 5 consecutive pipes, when there would be a triangular fracture in the next pipe, and the cracks in the next few pipes would be in a different line than the first section. This uniformity of alignment of the cracks indicated the rigidity of the joints.

Mr. Bowers. — How many cracks would there be in the pipes generally?

Mr. Potter. — Usually four.

Mr. F. L. Fuller. — I should like to ask Mr. Potter whether he thinks that the cracks in some of the pipes were due to the starting of a crack in an adjoining pipe. For instance, when he spoke about there being several hundred feet in length of cracked pipe, the question occurred to me whether the crack throughout the entire length might not be due to one weak pipe which cracked and communicated a strain to the adjoining pipes.

Mr. Potter. — My impression is that probably 85 per cent. of the pipes were broken by strain transmitted from defective conditions in the other 15 per cent.

Mr. G. C. Dunne. — Was every piece of pipe inspected before it was laid?

Mr. Potter. — Yes, it was; carefully.

Mr. George Bowers. — Do not most of these broken pipes hold their form pretty well? They sag somewhat, but you have a pretty good sewer even if it is cracked, don't you?

Mr. Potter. — Yes, except as to leakage. I recall but few cases where the diameter changed greatly. Measurements of the greatest and least diameters were taken after the breakage occurred. They varied in most cases three fourths of an inch to one inch. Of course it is impossible to say whether this difference in maximum and minimum diameter was attributable entirely to the cracking, because, as you all know, the pipes

are not truly cylindrical to start with. It is the practice, for instance, to accept a 24-in. pipe that may vary in maximum and minimum diameter one half an inch. So that it was impossible to tell in any particular case the exact deflection due to cracking.

MR. A. T. SAFFORD. — I should like to ask Mr. Potter what kind of joint he used and how it was made.

MR. POTTER. — We used several kinds of joints. The cement joints were made as follows: A gasket of tarred oakum was rammed thoroughly into the annular space, and the remainder of the annular space was filled with cement flush with the face of the bell. Another gasket of tarred oakum was then rammed thoroughly into the cement 0.50 in. inside the face of the bell. Then we finished off with cement mortar in the ordinary way, giving the proper bevel.

We also used a pipe joint made up of tar and cement, using it quite freely in the beginning of the work because previous experience on a 15-in. outlet was so satisfactory. We took a batch of Portland cement and mixed it with North Carolina pine tar and kneaded them as you would dough, sufficient for the joint. The pipe-layers, with their fingers, would then push the material in as far as they could and then with a proper calking tool force it in further, first putting in the tarred oakum gasket. The difficulty experienced with this joint on all large sizes was that before the material could get an initial set there was enough plasticity in the material to cause it to sag down from the top of the pipe, and no matter how carefully the spigot end of the pipe was supported so as to prevent its sagging, the mass of the material in the upper part of the annular space sank away and separated from the inside of the top of the bell, leaving, oftentimes, a space of 0.25 in. in an hour or two after the joint was made. On the smaller sizes, the objection to the tar cement joint is that if you do not use enough dough to make a complete joint there oftentimes will be no bond between the two applications, and a leakage will occur at the point of union. While tar cement makes a very good laboratory joint it is not wholly successful in practice for the reasons given. It gives an elastic joint, but the advantage of that elasticity in wet work is offset by the tendency to leakage in the larger pipes.

Toward the end of the Joint Trunk Sewer work, and on the laterals and extensions now under construction, we are using a sulphur-sand joint, — a mixture of sand and sulphur, — and with its use we find the leakage is materially reduced. It is applied just as lead is used in laying a water-pipe, without, of

course, the calking of the joint. We started to make this mixture on the ditch, using quicksand found in place, but it was so easy for the workmen to use much more sulphur than sand, because it poured the joint more readily, that we abandoned mixing it on the ditch and prepared it before bringing it on the work, regulating the proportions according to the season of the year. In winter we use more sulphur than in the summer, because with the same amount of sulphur the mixture flows more readily in summer. At low temperature, say from 0 to 35 degrees fahr., 35 per cent. sand and 65 per cent. sulphur pours as easily as 45 per cent. sand and 55 per cent. sulphur does in the summer time, and exhibits the same strength. We found, also, that while the tensile strength of sulphur was 80 or 90 lb. to the sq. in., the tensile strength of the mixture was as high as 850 lb. to the sq. in. within the mixture limits named. With higher percentages of sulphur than those named, the tensile strength rapidly diminishes. While in the winter time you do not get an absolutely tight joint, you get so nearly a perfect joint that it is altogether out of proportion to the results that the ordinary cement would give, made at the same time. The reason why this mixture does not give a perfect joint in the winter is due to hair cracks caused by its rapid cooling in coming in contact with the chilled pipe. A head of 10 ft. applied from the inside gave a much greater rate of leakage than desirable. In practice this pressure is never applied from the inside.

MR. F. L. FULLER. — With this sulphur cement is any gasket used?

MR. POTTER. — Yes, that was necessary; otherwise the sulphur would run inside the pipe.

MR. H. P. EDDY. — It might be interesting, Mr. Potter, to tell us how you control the heat in making the sulphur joint.

MR. POTTER. — We use gasoline furnaces and regulate the heat in the usual way. It is an easy matter for a workman who is accustomed to the work to keep the mixture at the proper fluidity. There is no trouble about that, but a man should be in constant attendance. When it cools and it is necessary to heat it quickly, the addition of a little sulphur — a handful, not more — brings the mixture very rapidly to the pouring condition.

MR. EDDY. — Is there a tendency to get it too hot, so that it won't flow?

MR. POTTER. — Of course that occurs occasionally. When you get the sulphur too hot it gets thick and pasty and the batch should be thrown out to cool and remelt; but we have lost very

little sulphur in that way. We are now using for the mixture a sand that looks almost as fine as flour. As stated before, we started to use the quicksand, which abounds in the section in which we were working, and the laboratory results from the local quicksand were as good, and sometimes better, than those secured from what we are now using. However, the lack of uniformity in the ditch material gave unsatisfactory results which warranted the additional cost, if any, of the manufactured article called posite.

A MEMBER. — How much does the material cost?

MR. POTTER. — The mixture is sold for $40 a ton, — 2 cents a pound, — while pure sulphur is sold at about the same price. The amount of sulphur wasted in mixing it at the site of the work warrants the payment of the same price. With 8-in. pipe we find it takes 2.5 lb. of the mixture to the joint. Whereas, counting the actual number of joints laid with a barrel of pure sulphur, the cost amounted to the same. On account of the novelty of the use of that material, contractors did not know what it was worth, and I therefore established a price of 5 cents per foot to cover the extra expense. On pipe from 12 in. to 15 in., I have established a price of 6 cents per running foot. That is over and above the price of laying, which would ordinarily include the cement. But the contractors, knowing they are paid for this extra, cut out practically the cost of cementing the joints in their bids. Hereafter I think I shall let contractors bid for it direct.

A MEMBER. — Is water troublesome in making these joints?

MR. POTTER. — The advantage of the sulphur in the wet trench is this, that in all sewers from 8 to 15 in. it is possible to join two or three and sometimes four joints together on the bank on temporary cradles and lower them into the trench as one pipe; and before making the ditch joint, it is possible to thoroughly bed the section lowered down. Having that section thoroughly bedded, you can dig out a satisfactorily deep bell hole without disturbing the joints, which is impossible in laying a pipe at a time with cement, because in making the proper bell hole you disturb the joints and affect the alignment of the pipe as well. We have been able to get a bell hole sufficiently large so that we can properly handle the water at the joint. You can let the water rise half a minute after you pour your joint.

A MEMBER. — Don't you think digging deep bell holes has a tendency to increase settlement?

Mr. Potter. — By a deep bell hole I mean a hole that is free of the bell. It is ordinarily difficult to get even that through bad material. But it is not more than six inches below the bottom of the pipe.

A Member. — How long does it take for the sulphur joint to become fairly hard?

Mr. Potter. — About a minute.

A Member. — Is it as hard then as it will be?

Mr. Potter. — No, it is not. I have not tested the mixture a minute after pouring, but I have tested it five minutes, and in five minutes it gets about 80 per cent. of its strength.

A Member. — Is there trouble from the sulphur taking fire?

Mr. Potter. — No, we have had that trouble, but when it starts to burn it can be put out with sufficient of the powdered material to cover the burning portion.

A Member. — Does the pipe crack just as badly with the sulphur joints as with other joints?

Mr. Potter. — No. There were no cracks, because when sulphur was used we were laying it either through quicksand or where quicksand was very close, and after laying the sulphur joints we bed it in quicksand, putting water in the ditch. This packs the sand so thoroughly around the pipe that there is no further tendency to settlement. On the rest of the work concrete was used.

Mr. Leonard Metcalf. — I would like to ask about the platform of which you speak. Did I understand you to say that the planks upon which the pipes were laid were separated by a space of 6 or 8 in.?

Mr. Potter. — There were 2 in. spaces between them. We used 2 in. by 4 in. stuff laid flat and spaced 6 in. centers.

Mr. Metcalf. — There were four of these?

Mr. Potter. — Generally; the number used depended on the size of the pipe and width of the trench.

Mr. Metcalf. — With cross pieces 18 in. apart?

Mr. Potter. — Yes; each 3 ft. length of pipe had two supports.

Mr. Metcalf. — And these were sawed out in the form of a cradle?

Mr. Potter. — No; these cross pieces were securely nailed to the longitudinal pieces before lowering them into the ditch, and the pipe was held in place by wedges nailed to the cross pieces.

Mr. Metcalf. — Were the wedges nailed on before being put down into the ditch?

Mr. Potter. — No; the wedges were nailed on afterwards.

Mr. Metcalf. — So that the pipe simply rested on these cross pieces and on the wedges?

Mr. Potter. — Yes; each pipe resting on two supports until the backfilling was completed.

Mr. Metcalf. — Then did you make a practice of having the water flow outside of those timber supports?

Mr. Potter. — It could flow between, outside and over the lower portion of the timber supports.

Mr. Metcalf. — As a matter of fact, did it flow between or outside of them?

Mr. Potter. — The bulk of the water flowed between the planks.

Mr. Metcalf. — I asked the question because I first tried the method of laying sewer pipe in a wet trench upon a plank out at Concord a few years ago, and we found that it worked there very satisfactorily. Of course, there are two general methods of laying on plank, — one, laying your timber directly under your pipe; the other, laying two or more timbers along the sides of the pipe and laying your pipe directly on a block, bridging from one stringer to the other. It has always seemed to me that the safer construction was to lay a single timber underneath the pipe and then to lay the pipe on a cradle laid on that, for the reason that the water tends then to flow outside and away from that single timber; then you have your water flowing between that and the sheathing along the sides of the trench, whereas, in the other case you have the water flowing directly under the middle of the pipe and tending to wash out the supporting material. Moreover, it has always seemed to me that in laying two timbers, one on each side of the pipe, rather than one or more directly under the pipe, you have the further disadvantage of bringing your supporting timbers nearer your sheet-piling supporting the wet trench, which must result, if the sheet-piling has been driven below the level of the bed of the trench, in greater settlement if this supporting timber is close to the sheet-piling than if it is in the middle of the trench. In the work of which I spoke, we aimed to keep the water toward the sides of the trench. We found in quicksand, for instance, where the material was very fine and running very freely, that, where the material was very troublesome to the men in the excavation, so that they would sink above their ankles in the mud, and it was soft and boggy, after the plank had been rammed in place and allowed to remain for a short

time the trench was perfectly firm and the men could walk upon it without disturbing the bottom of the trench. Our practice there was to use a 3-in. plank of the width of the inside diameter of the pipe. So far as I know we have had no trouble with settlement since that sewer was constructed. I presume we did have some cracked pipe. The pipe was too small to crawl through. I know we did have some leakage, but the joints were made there with the ordinary cement, bedding on small blocks sawed off and placed in the pipe to preserve the alignment, using Portland cement and allowing the joints to set until they were hard before any backfilling was done. And, as far as I now recollect, we did not have any difficulty of a serious nature in backfilling about that pipe. In one or two cases, where it was left overnight not properly braced, the pipe floated and had to be relaid.

Mr. Potter. — The advantage that I thought we had in using a cradle over the single timber was this: In attempting to maintain a drain at either side of the pipe in a quicksand ditch, it was difficult to prevent the material from slumping in on the sides, thus forcing the water to flow over the plank. With the cradle construction you have the spaces between the 2 in. by 4 in. for the water to run under the pipe. Of course I realize that if we used an underdrain, the trench in the middle, draining into an underdrain, might cause the material around the pipe to wash into the underdrain, undermining the support. Having no underdrain the cradle has an advantage over the single plank.

Mr. E. W. Branch. — How much water can be taken care of in that way?

Mr. Potter. — We have taken care of the water from three or four hand-pumps.

Mr. George Bowers. — Was the sheath-piling drawn after the trench was filled?

Mr. Potter. — The sheathing was drawn after the trench was filled, when it was not driven below the pipe. Sheath-piling was never drawn until we had 4 or 5 ft. of earth over the pipe.

Mr. Bowers. — Don't you think that may have something to do with the cracking of the pipe?

Mr. Potter. — I am not prepared to say, and I doubt whether any engineer is prepared to say exactly what occurs when you pull sheathing and just how far the pulling of sheathing is responsible for ruptured pipe.

MR. BOWERS. — Where you left the sheath-piling in, did the pipe crack? I mean where you sawed it off and left it in?

MR. POTTER. — Yes; in one or two places I know that it cracked, but in the majority of cases where the cracks occurred the sheathing had not been left in.

MR. BOWERS. — Don't you think it is safer to leave it in?

MR. POTTER. — Unquestionably I do. But sheathing is rather an expensive item. An engineer in the attempt to save money for his clients takes certain chances. For instance, we had some 10-in. sewer laid in drift formation about 15 ft. deep. There were two points on this section where the pipe had to be relaid. Looking through the manhole I could not get a light through it and so I ordered the contractor to take it up. In both cases we had removed the sheathing, which had not extended down below the pipe. Of course we believed it was the fault of the contractor. But my own personal conviction now is that, although we took great care in pulling the sheathing, the fault lay in pulling it. On this particular section the contractor's price on sheathing was very high, — $35.00 per thousand, — and he wanted every foot of it left in. Whenever we ordered it taken out, even where it was perfectly safe to do so, he gave us a written protest. It was a rather unpleasant position to be in, and can be avoided by fixing the price to be paid for sheathing, which I now do.

MR. BOWERS. — To what depth below the pipe was the sheathing driven?

MR. POTTER. — Whenever it was necessary to go below the pipe the sheathing was not pulled. Sometimes we had to go 2 ft. and in one instance 3 ft. below. This drift formation — I presume you have some of the same formation here — is a mixture of clay, sand, bowlders and water. In small pipe work, the bowlders interfere with your sheathing, and in hammering or digging them out you loosen the material back of the sheathing, forming a cavity which is constantly enlarging. In one instance, in a 12-ft. ditch, the contractor experienced a great deal of trouble in getting the sewer pipe laid. In a stretch of 100 ft., I think, he spent two weeks in reaching sub-grade, and just before he was ready to lay pipe, the street dropped in from curb to curb, a drop of 2 ft. This illustrates the nature of the material encountered. When you pull your sheathing you don't know what new condition is brought into play. It may be a large cavity or a very small space. You don't know what to expect.

Mr. Metcalf. — What was the depth of excavation below the bottom of the pipe in the rock cuts?

Mr. Potter. — We aimed to have our rock-cut 6 in. below the bottom of the barrel — 4 in. or more below the bell of the pipe.

Mr. Metcalf. — And the material was of such a character that in backfilling it could be thoroughly rammed under the pipe?

Mr. Potter. — Yes. As I said before, it is impossible for an engineer or his assistant to be on the spot all the time. In one or two places I personally supervised the relaying of the pipe where cracks occurred, and I knew that in one or two cases the rock at points was but 1.5 or 2 in. away from the barrel of the pipe, making it utterly impossible to get the backfilling properly in on that side, thus producing an opportunity for starting a line of broken pipe.

Mr. A. H. French. — I would like to ask the speaker if he has had any experience in inclosing sewer pipe in concrete for the purpose of securing water-tightness?

Mr. Potter. — If you depend upon the concrete for water-tightness you cannot get very good results. The joint itself has to be made tight. All that you can expect from the concrete is a stable foundation. I have surrounded pipe with concrete — in repairing some of the breaks, for instance; when I saw that the pipe was in good condition I did not remove it, but simply surrounded it with a lean concrete, perhaps about 1 of cement to 12 of gravel; but this did not reduce the leakage at that point as much as I had expected.

Mr. French. — As I understand you, you do not think it would have been successful, even if you had used a very rich concrete?

Mr. Potter. — I think it probably would have reduced it somewhat more.

Mr. French. — Some years ago I had occasion to lay a pipe sewer where the ground water was to be from 8 to 10 ft. above it, and where, to secure water-tightness and additional strength, the pipes were inclosed in a rich concrete about 6 in. in depth, care being taken to get as good joints as possible with Portland cement. It was fairly successful, but not entirely so.

Mr. Potter. — I have not used concrete extensively around pipe except in covering the broken pipes, with the hope and expectation of reducing the leakage due to cracks. There has been some reduction. I have used concrete around the pipes on

deep trenches where I was unwilling to risk the standard thickness of pipe alone.

Mr. Eddy. — Mr. French, have you had any experience with cracked pipe?

Mr. French. — With our 50-odd miles of sewers in Brookline my only experience with cracked pipe is in one or two instances where pipe was laid in filling which settled. There may, of course, be much broken pipe in our sewers, but we have no knowledge of it.

A Member. — Has anybody ever crawled through to see?

Mr. French. — My belief that there is not much broken pipe in our system rests on the fact that we almost never have had any pipe collapse.

A Member. — They may be cracked and badly cracked and not collapse.

Mr. French. — I have known of a few instances where pipe has been broken by having its vertical diameter much reduced and its horizontal diameter correspondingly increased and to still serve as a conduit. One of the most unfortunate cases in pipe breakage that ever came to my knowledge occurred many years ago, where a 24-in. Akron pipe was used as a conduit to convey water from a filtering gallery some 4 000 ft. through a swamp to a pumping station. The soil was a quicksand, so that the trench was boxed and the pipe laid on a timber floor and held in place with blocking. Shortly after the pumps were started it was found that the swamp was being drained and that the water supply was very much discolored. Investigation showed that most of the pipe was broken, as Mr. Potter has described, into four quarters, breakage following the top, bottom and two sides. A timber flume was then placed on the same line as the pipe, care being taken to get as tight work as possible, but as this was unsuccessful, the pumping station was moved over to a point near the filtering gallery and an iron force main laid through the swamp.

Mr. Patterson. — We have had in Revere very little trouble with pipe breaking. In a sewer constructed last year, 18 in. in diameter and nearly three quarters of a mile in length, laid through a marsh on a solid clay bottom, we had very little trouble. The pipe was large enough so that a man went inside to caulk it up and found but very few leaks.

A Member. — You laid that on a natural foundation?

Mr. Patterson. — We laid it on very fine gravel or sand.

It ran through marsh land, so that at high tide we had 8 or 10 ft. of water over it.

Mr. Eddy. — I wonder whether Mr. Dunne, of the Portland Stoneware Company, has any excuse to offer for all this cracked pipe we have heard about to-night?

Mr. Dunne. — I don't know that I have, because our experience has been entirely in New England, and in an experience of 35 years we have had very little cracked Portland pipe. I wanted to ask Mr. Potter one question in regard to a matter which I am not able to explain from a manufacturer's standpoint. You can pile up a lot of pipe in your yard over winter, after inspecting it carefully and finding it all sound. In the following spring you will find a lot of it cracked. I don't think any manufacturer has been able to explain that. The only possible explanation I was ever able to get came from the late M. M. Tidd. Mr. Tidd told me he knew for a fact that iron water-pipe cracked three months after it was laid, and his explanation was that the pipe was cooled too quickly in the making. He believed that the same reason might apply to vitrified glazed pipe, and on that assumption our company, instead of taking its pipe out of the kiln in 10 or 12 days, never removes it within 15 days or 16 days. The result is that we have less cracked pipe than we formerly had. Whether that is any solution of the problem or not I don't know.

Speaking of the question of pulling sheathing, I knew a case which occurred a few years ago in Medford, where 18-in. pipe was cracked in a section where the sheathing was removed, while in a similar section on the same work where the sheathing was not pulled the pipe was found to be sound.

Mr. Eddy. — It has been our experience in Worcester that a very material proportion of pipe that has been piled out of doors through the winter is found cracked in the spring. I don't know that I ought to venture to give any percentage, but I should say it would be more than 25 per cent., especially in the larger pipe; not so much on the smaller.

A Member. — I have had a good deal of pipe crack that way. We had a lot of 20-in. pipe left over winter, and it was in very poor condition in the spring. My explanation is that the large sizes especially are not usually well glazed at the ends, allowing a certain amount of moisture to get into the pipe. Under these conditions, the freezing and thawing during the winter seem to me enough to explain the cracking of the pipe.

Mr. Dunne. — In the manufacture of pipe the spigot end

sits in a socket, making it practically impossible to glaze that end. Some of you will recollect an article written by the late Mr. Chesbrough in one of the engineering papers, in which he claimed that the only proper sewer pipe was pipe that was unglazed inside the socket and for two inches outside the pipe at the spigot end, because it would hold the cement better. But it is impossible to make it that way.

Mr. Potter. — I saw the same effect at Cincinnati, a few years ago, on shale paving brick. I went out with Mr. Bouscaren, the engineer, to examine the uncompleted reservoir. Mr. Bouscaren was a very careful engineer in the selection of his material, and he had rejected the brick from two or three concerns, finally deciding upon one which he claimed was superior to all others. These bricks were delivered in the fall. We were there in the spring, and there was hardly one of these bricks, which were lying around the ground, that was not cracked. These cracks were irregular, but extended all over the surface of the brick, some of them penetrating the brick about half an inch or more. That these cracks were due to the same cause as has been referred to in the case of the pipe would seem probable.

Mr. Dunne. — You have referred to a test at Liverpool. Perhaps you are aware of the fact that double-strength pipe in this country is called standard in England, as far as the thickness is concerned.

Mr. Metcalf. — I was rather surprised when Mr. Potter spoke of the experiments made by some Liverpool engineer showing that pipe had very little elasticity. I remember having a talk with Mr. T. Howard Barnes, in which he spoke of certain experiments made by him in Medford on the elasticity of pipe, and his results were quite different, but I don't remember the details of the test.

Mr. Dunne. — There is such a thing as burning the pipe too hard. The moment it is burned too hard it becomes brittle.

Mr. Eddy. — Perhaps Mr. Taylor can give us some enlightenment on this subject.

Mr. Taylor. — I don't know that I have very much to say, but speaking about cracked pipe my own experience has been, as I remember it, that pipe that has cracked in the pile is almost invariably cracked on the lower side. I have the impression that it is caused by the absorption of a certain amount of moisture and frost and ice, and possibly, also, by the weight of pipe on the upper side if it is lying in a pile. I don't know that I have had any particular trouble with small sizes in that way.

In relation to the pulling of sheathing, I did have experience on a trench 40 to 45 ft. in depth, where we used three sets of sheathing. At first we left the sheathing in, afterward cutting it off at the top of the sewer, — a brick sewer of 12-in. brick work, — and no cracks appeared. Afterward the material improved somewhat, at least the level of the ground water was reduced, and we pulled a certain amount of sheathing, using the water jet with each plank we pulled, with about a 0.75-in. pipe, putting it into the space from which the plank was pulled and filling the space carefully with sand. The material was coarse sand and fine gravel,— dry down to within perhaps a foot or two of the bottom of the sewer. We tried this method repeatedly, but, in spite of all that, the sewer cracked, — not very much, but it opened on top a little. The first indication we had of it came during a high tide. I had the bulkhead in the sewer of 16-in. brickwork. The first thing we knew came a high tide and we had 4 ft. of water in the trench and the brick masons had to come out. We inspected it and found that the sewer had spread about half an inch on each side, letting the water into the trench. We found it was impossible to pull the sheathing and not have it crack, so afterwards the lower set of sheathing was left in, cutting it off level with the top of the brick work. After that there were no cracks as far as I know, anywhere in the line of the sewer. The sewer was of 12-in. brick work and under a considerable head of ground water after it was completed. In one section of the work the ground water was about 15 ft. above the crown of the arch, or 20 ft. above the invert. The sewer was plastered on the crown and where it could be got at in the invert. A considerable amount of leakage did come in through the brick work — not in any visible stream, but oozed through so that in a length of over 4 000 ft. there were perhaps 6 in. in depth of water. There was no spot where it spurted through; it simply oozed in from the pressure from the outside and came through the pores of the brick.

I never had very much experience in the matter of cracked pipe, and have not had occasion to look into that matter very much. I know it is difficult to make water-tight joints and water-tight sewers. We try to do so, but very rarely accomplish it, but of course we do get by various methods what are practically water-tight joints, and I think the percentage of leakage in sewers constructed during the past few years has been very materially reduced.

I was connected a good many years with the Worcester

sewers, and did a large part of the construction there, and always in wet ground in the first few years we left the lower joints open with the intention of lowering the ground water. Mr. Eddy knows, I have no doubt, whether many of these are giving a great deal of trouble to-day. In some of them there may be a certain amount of material displaced under the pipe. In some of the localities I should think the settlement might be considerable. Whether this has taken place or not I do not know. There are places where pretty fine sand occurs, and it seems to me the material must flow into the sewer, and that would continue until a certain amount of coarse material had become stable and filtered the water without carrying the sand with it. I think some of these sewers may have been replaced, though Mr. Eddy knows more about that than I do.

Mr. Potter. — In my paper I made reference to the fact that I noticed that in a number of instances the sewers where these cracks were discovered were examined immediately after a rainstorm. To my mind, that would indicate that in these points the cracking of the pipe was caused by shock. We all know that while sewer pipe has many excellent qualities it has not the quality of withstanding shock. I should like to know whether any gentleman present has had any opportunity of observing pipe in trenches at such a time. We all know that after a rainstorm a newly filled ditch will settle. Oftentimes the settlement is not uniform, causing a shock which does not occur with a gradual settlement.

Mr. Eddy. — Mr. Taylor has brought up an interesting matter which I was very much in hopes no one knew anything about. We have in Worcester a few sewers which were laid without much cement in the bottom. But so far as I know there is not one of them which is giving, or ever has given, any trouble from admitting sand. I have in mind a brick sewer laid on Highland Street, in the vicinity of Seaver Street, through a locality where there is considerable fine sand. As I understand it, there are four courses of brick laid without a particle of cement in the joints. Sometimes you can see water coming through these joints, although not in any great amount, which seems to me a proof that the joints have closed themselves to quite a large extent. I have within a few days been making some analyses of our measurements of flow, and I find that the flow from our system, comprising 130 or 140 miles, — that is, the sanitary and combined systems, — does not yield, on the average, to the best of my knowledge and belief, a large amount of

ground water. I think the figures range from a minimum of perhaps 10 000 gal. per mile up to a maximum of from 30 000 to 50 000 gal. per mile. Those figures do not represent the maximum for any particular day, but they are figures which cover certain days of every month in the year, corresponding, perhaps, as nearly as you can possibly get at it, to the normal leakage in these sewers; which, it seems to me, considering the age of the system — part of it is about 40 years old — is a fairly good record. About 60 miles of sewers, I believe, are composed of cement pipe. This has stood the test of time remarkably well. Some of that was laid back in the early sixties, and is now 40 years old. That, I think, has done pretty well for cement pipe, and in those days it was made with Rosendale cement. We are old-fashioned enough to be laying cement pipe to-day. A large proportion of our surface-water sewers are made of Portland cement pipe, and an excellent pipe it is, — absolutely straight and round and strong. It is a little heavy; one of the worst objections to it is that it is heavy to handle. But it has given remarkably good satisfaction, and I know of no good reason for not using it.

A Member. — What is the cost of it?

Mr. Eddy. — The cost of that pipe, delivered on the work, is the same as the other delivered on the car. It is made in Worcester by a local concern. They are now making Portland cement pipe.

A good record can be kept of sewers by the drain-layers, the men who make the connections. The drain-layer gets a chance to inspect the main sewer when he makes a connection, and, if a record is kept of these inspections, a pretty good idea will be gained, after a few years, of the condition of your system as a whole. We have done that to a considerable extent. We have some cracked pipe, but I think we haven't very much. In 13 years we have only relaid one small section of pipe sewer because of breakage, and that breakage was apparently owing to the settlement of the fill through which the sewer was laid several years prior to its failure. Although we may have some cracks, the pipe has not given out entirely. Perhaps Mr. Marble, of Lawrence, can tell us what his custom is in making cement pipe joints.

Mr. Marble. — We have made joints usually with Rosendale cement. We have one case of a sewer about 2 000 ft. in length, going to pieces entirely, so that we had to relay it. But it was in wet ground and possibly wasn't properly backfilled. I

think the trouble was with the size of pipe and with the ground into which it went being so wet. I haven't much faith in the ordinary 24-in. pipe standing up in wet ground. I would like to ask Mr. Potter about those cradles, whether the 2 in. by 4 in. timbers were laid on edge or flat?

Mr. Potter. — Flat.

Mr. Marble. — And then the pipes are joined together and laid on this cradle on the bank. I cannot understand how you lower them without breaking the joints.

Mr. Potter. — I stated that the pipe is laid on temporary cradles on the bank. The permanent cradles were placed in the bottom of the ditch before the pipe is let down. In reference to breaking the joints in lowering the pipe, our experience is that the sulphur sand joint is so strong the men can handle it quite roughly without any breaking. We rarely broke a single length. In lowering a section in which we had T's and Y's, more care had to be used, because if we lost a T we had to make the whole joint over again. We broke the bell of the T occasionally, but rarely the barrel of the pipe.

A Member. — I would like to ask Mr. Potter with regard to that sulphur joint, whether the mixture is difficult to make.

Mr. Potter. — No, it is not. We have the commercial product prepared at the Bergen Point Sulphur Works, with a special sand.

A Member. — Isn't it necessary that the bell and spigot of the pipe should be perfectly dry when the sulphur is run?

Mr. Potter. — No. All we do in the case of a ditch joint is just to sweep it out. A man swabs it just as he is ready to do the pouring. No pretense is made to have it absolutely dry. The heated material will absorb the moisture inside of the bell and the dampness doesn't seem to affect the water-tightness of the joint. We have laid the joint in water, holding the jointer the full height on one side and 3 in. on the other. The pouring is started on one side until the sulphur forces the water out from the lower opposite side of the joint. Then when we had the bottom fixed, we put the jointer on the other side and completed our joint.

A Member. — What is the leakage through sulphur joints in 6 in. or 8 in. pipes?

Mr. Potter. — Between 4 000 and 5 000 gal. per mile per day would be a fair average on work so far constructed.

Association of Engineering Societies.

Vol. XXXV. AUGUST, 1905. No. 2.

PROCEEDINGS.

Montana Society of Engineers.

Butte, Mont., September 9, 1905. — The regular meeting of the Society for September was held in the Society room at the usual hour, with Treasurer Barker in the chair. After the arrival of a quorum the minutes of the previous meeting were read and approved. The Chair appointed the following Committee on Nomination of Officers for the coming year: Eugene Carroll, Chas. W. Goodale, Frank L. Sizer. The evening was devoted to the discussion of various geological features found in Montana, as well as electric railways as a means of cheap and rapid transit. Dr. Musigbrod and Dr. Borgnis were the leaders of the discussion. After a very interesting meeting the Society adjourned.

Clinton H. Moore, *Secretary.*

Civil Engineers' Club of Cleveland.

Regular Meeting, September 12, 1905, at Clifton Club. — Meeting called to order by President Green; present, sixty-three members and visitors. Reading of minutes was dispensed with owing to the social nature of the occasion. Applications for active membership, approved by the Executive Board, from the following were read: Walter E. Dalgleish, Charles E. Hadley, Robert T. Kent and Jesse M. Woodward. A Committee on Resolutions on the death of Mr. Charles H. Wellman, consisting of the following members, was appointed by the President immediately after that sad event: Mr. John W. Seaver, Mr. John McGeorge and Mr. J. D. Cox. Owing to absence from the city, Mr. Cox was unable to serve and Mr. McGeorge submitted the report of the committee, signed by himself and Mr. Seaver. The report of the committee, hereto appended, was unanimously adopted and the secretary instructed to send copies of the resolution to Mr. Wellman's family.

The Secretary announced the presentation to the Club of a fine group picture of the American Society of Civil Engineers by Mr. Charles Paine, first president of the Club. In this connection, the following resolution by Mr. M. E. Rawson, was unanimously adopted:

"*Resolved:* That the Secretary be and he is hereby authorized and directed to extend the thanks of the Civil Engineers' Club of Cleveland

to Mr. Charles Paine, its first president, for his gift of a handsome group picture of 'Members of the American Society of Civil Engineers, in 1905.'"

The Secretary read a communication from the Secretary of the American Society of Civil Engineers announcing the adoption by that body of a resolution of thanks for courtesies extended by this Club on the occasion of the annual meeting, June 20–23, last.

The Secretary announced the appointment, by the Executive Board, of Prof. C. H. Benjamin, to serve as the representative of the Club in the preparation of so much of the new Building Code as relates to boiler inspection and smoke abatement, this appointment being made in pursuance of a request of the Building Code Commissioner, Mr. John Eisenman.

The Secretary also announced the taking of membership in the American Society for Testing Materials, for the Club.

Adjourned.

Preceding the business session, dinner was served those present in the Clifton Club dining-room, and the latter part of the evening was devoted to dancing.

JOE C. BEARDSLEY, *Secretary.*

Report of Committee on Resolutions on the Death of Charles H. Wellman.

Whereas, In the death of Charles Henry Wellman, June 21, 1905, this Club has suffered a great loss, and we desire to place on record our appreciation of his high character, not only as an engineer, but as a man,

Therefore, be it resolved, That we tender to his bereaved family and his associates in business our sincere sympathy, and that a copy of this resolution and the accompanying biographical sketch be spread on the records of this Club.

JOHN W. SEAVER.
JOHN McGEORGE.

EXCURSION TO CONNEAUT HARBOR, SEPTEMBER 30, 1905. — One of the pleasantest and most profitable trips the Club has ever enjoyed was that of the 30th ult., to Conneaut. Through the courtesy of Mr. Johnston, general superintendent of the New York, Chicago & St. Louis Railroad, a special car was attached to train No. 6, leaving Broadway Station at 8.20 A.M. and arriving at Conneaut at 10.57 A.M. Forty-eight members (including three who joined us at Conneaut) took advantage of this opportunity. Lunch was served at the Cleveland Hotel promptly on our arrival, transportation from the railroad station to the hotel by special trolley car having been provided by Mr. Richardson, superintendent of the Dock Company. The same car was in waiting to convey us to the harbor immediately after lunch.

At the harbor, Mr. Richardson, seconded by his assistants of the Dock Company, took us in charge and gave us three hours of lively sight-seeing that included everything there was to see about the entire harbor, — at least if anything got away from anybody, it wasn't Mr. Richardson's fault, or that of his assistants, Messrs. Fickinger, Walker, Wyman, Buss, Curtiss and Meek. First, a general view of the harbor, both inside and out, was taken from the deck of a Great Lakes tug. Then a landing was

made at the Hulett machines farthest down the river, which were unloading one of four steamers that had just arrived at noon.

The big bridge tramway plant erected by the Brown Company for the storage yard was also inspected. This was in operation, conveying ore from the front to the back of the yard. Four Brownhoist fast plants were also engaged in unloading another of the four steamers.

From here, the cable house, power plant and car-weighing plant were visited in succession, and then the McMyler fast plant on the west side of the slip. It is noticeable that a large proportion of this machinery was designed and erected by members of this Club.

A small number also visited the water-works pumping plant, which also includes a Jewell filter. The machinery here was antiquated and in rather bad order, but is soon to be replaced by a modern high-duty crank and fly-wheel pumping engine.

The return trip to town was commenced at 4 P.M., after taking leave of our hosts who wound up our entertainment by distributing cigars of a particularly good brand.

An excellent dinner was served at the railroad restaurant at 4.30 and our departure taken at 5.10 P.M., arriving in Cleveland at 7.40.

JOE C. BEARDSLEY, *Secretary.*

REGULAR meeting, October 10, 1905, at Club rooms.

Meeting called to order by President Green. Present, fifty members and visitors. Minutes of two preceding meetings read and approved. The tellers, Messrs. Herman and Wight, reported the election to active membership of Walter Edmund Dalgleish, Charles Ethan Hadley, Robert Thurston Kent and Jesse Marion Woodward.

The appointment of the following committee by the President to consider the advisability of amending the constitution to permit the formation of sections for the consideration of special branches of engineering was reported from the Executive Board: Beardsley, Nelson and Lane.

Professor Benjamin, representative of the Club in the preparation of certain sections of the new building code, reported that he had been informed by the Building Code Commissioner that he would be notified when those sections were ready to be taken up, but that he had not as yet been so notified.

Mr. John McGeorge called the attention of the Club to certain inconsistencies of the new building code, especially with regard to the use of reinforced concrete in building operations, and moved the appointment of a committee to confer with the commissioner relative to changes considered desirable. Mr. Ritchie seconded the motion and also moved as an amendment that any changes considered desirable by the committee be first reported to the Club before they were recommended to the Buildin Code Commissioner. The motion as amended was unanimously adopted, and the President named the following committee: Benjamin, McGeorge and Barnum.

The following resolution by Mr. Herman, seconded by several members, was unanimously adopted: "*Whereas*, the inspection trip of the

Civil Engineers' Club of Cleveland to Conneaut Harbor on September 30, 1905, was most enjoyable and instructive, and the Club wishes to express its thanks to Mr. A. W. Johnston, general superintendent of the New York, Chicago & St. Louis Railroad, for the special car so kindly furnished by him; and to the gentlemen connected with the Pittsburg & Conneaut Dock Company, viz., Messrs. R. R. Richardson, superintendent, and his assistants, P. J. Fickinger, C. Walker, J. M. Wyman, H. H. Buss, M. E. Curtiss, G. S. Meek and others, for their untiring attention to the members in showing and explaining the working of all the appliances and structures of this magnificent plant, as well as in attention to the more material necessities; therefore, *Be it resolved*, that the sincere thanks of this Club are hereby extended to all of the gentlemen named in the foregoing, and the Secretary is directed to send to each of them a copy of this resolution."

The paper of the evening, "Engineering Ethics and Fees," was then read by Mr. F. C. Osborn and was discussed at length by Messrs. Lane, Herman, McGeorge, Boalt, Ritchie, Benjamin, Green and others.

At the conclusion of the discussion Mr. Ritchie moved the appointment of a committee to consider the advisability of formulating a code of ethics and scale of fees for this Club. Seconded by Mr. Palmer and unanimously adopted. The President afterward named on this committee Messrs. Osborn, Howe and Hopkinson.

Adjourned.

Joe C. Beardsley, *Secretary*.

Engineers' Club of St. Louis.

St. Louis, June 7, 1905. — The 600th meeting of the Engineers' Club of St. Louis was held at the Club rooms, 3817 Olive Street, Wednesday evening, June 7, 1905, President Flad presiding. Forty members and six guests were present.

The minutes of the 599th meeting were read and approved. The minutes of the 391st meeting of the Executive Committee were read.

The Secretary read a letter from Mr. Metzger, expressing his regrets for not being able to be present.

A letter from Mr. Maltby was read describing conditions on the Isthmus of Panama. Mr. Maltby reported that he preferred conditions there to those of the Mississippi Valley.

As this was the 600th meeting of the Club a special program had been prepared, and addresses and remarks appropriate to the occasion were given by Messrs. Holman, Bryan, Robert Moore, Pitzman, Ockerson, Fernald, Turner, Wall, Langsdorf, Geo. Johnson, Colnon, Van Ornum, Schwedtmann, Moreno, Pfeifer, Purdon and Greensfelder. Light refreshments and cigars were served throughout the evening.

Adjourned.

R. H. Fernald, *Secretary*.

St. Louis, September 20, 1905. — The 601st meeting of the Engineers' Club of St. Louis was held at the Club rooms, 3817 Olive Street, Wednesday evening, September 20, 1905, President Flad presiding. Twenty-six members and twenty-one guests were present.

The minutes of the 600th meeting and of the special meeting called by the President, July 28, were read and approved. The minutes of the 392d meeting of the Executive Committee were read.

Applications for membership were read from Raymond William Dull, Carl Oscar Nordensson, William B. Lemmon, Roy E. Peshak, Kurt Toensfeldt, William C. Weidmann.

An invitation was read from the University of Illinois asking the Club to be represented at the installation of Dr. James as president of the university. Mr. Bryan moved that the Club send a delegate and pay his expenses. The motion was lost.

Mr. Wm. H. Bryan's paper upon " An Efficient Modern Steam Plant in Flour-Mill Service," called attention to the low water rates in an engine recently tested, and brought out discussion by Messrs. Flad, Fish, Layman, McCulloch, Chafee, Humphrey and Swope.

Mr. Chafee called attention to the death of Mr. Blaisdell, a member of the Club, and moved that a committee be appointed to draw up a suitable memorial.

Adjourned.

R. H. Fernald, *Secretary.*

Boston Society of Civil Engineers.

Boston, September 20, 1905. — A regular meeting of the Boston Society of Civil Engineers was held at Chipman Hall, Tremont Temple, Boston, at 8.15 p.m., President John W. Ellis in the chair; twenty-nine members and visitors present.

The record of the last meeting was read and approved. Mr. Albert H. Lavalle was elected a member of the Society.

On motion of Mr. Blodgett, the thanks of the Society were voted to the following officials of the Old Colony Street Railway Company for courtesies extended to members of the Society on the occasion of the visit to the power station of that company this afternoon: George Seibel, general superintendent; John T. Conway, assistant superintendent, and R. F. Gammons, division superintendent.

The President announced the death of Mr. Dean C. Warren, a member of the Society, which occurred on July 6, 1905, and it was voted to appoint a committee to prepare a memoir. The committee appointed consists of J. Albert Holmes and DeWitt C. Webb.

Mr. George W. Blodgett read the paper of the evening, entitled " Recent Developments in the Old Colony Street Railway System." The paper was illustrated by lantern slides.

Adjourned.

S. E. Tinkham, *Secretary.*

ASSOCIATION
OF
ENGINEERING SOCIETIES.

VOL. XXXV. **SEPTEMBER, 1905.** NO. 3.

PROCEEDINGS.

Louisiana Engineering Society.

NEW ORLEANS, LA., JANUARY 14, 1905.—The annual meeting of the Louisiana Engineering Society was held with Mr. Frank M. Kerr in the chair in the absence of the president and vice-president.

The minutes of the meeting December 9, 1904, were read and approved.

The annual reports of the Board of Direction, officers and committees, were read and received.

The ballots for officers for the year 1905 were counted, and the following gentlemen elected:

President — W. B. Wright.

Vice-President — Prof. J. M. Ordway.

Secretary — G. W. Lawes.

Treasurer — Walter H. Hoffman.

Director — Ernest L. Jahncke.

Member of the Board of Managers of the Association of Engineering Societies — J. F. Coleman.

The newly-elected officers were then installed. In assuming the duties of president, Mr. Wright thanked the Society for the honor conferred, and said he would work for the welfare of the Society and use his best endeavors for its advancement.

The reading of the annual address of Mr. J. F. Coleman, the retiring president, which had been placed in the Secretary's hands, was postponed until the next meeting, so that Mr. Coleman could be present and read it.

Refreshments were furnished during the meeting and a pleasant evening was spent.

Adjourned.

G. W. LAWES, *Secretary.*

NEW ORLEANS, LA., FEBRUARY 13, 1905. — There being no quorum, no meeting of the Society was held.

G. W. LAWES, *Secretary.*

NEW ORLEANS, LA., MARCH 13, 1905.—The regular meeting of the Society was called to order by President W. B. Wright. Nine members and two guests were present.

The minutes of the annual meeting January 14, 1905, were read and approved.

The Chair announced the following committees:

Auditing: A. C. Duval, chairman, C. W. Wood, E. L. Jahncke.

Library: J. W. Armstrong, chairman, John F. Richardson, J. T. Eastwood.

Membership and Attendance: Alfred F. Théard, chairman, W. T. Crotts, Warren Johnson, J. F. Coleman, Gervais Lombard.

Advertising in JOURNAL: Ernest D. Ivy, chairman, A. M. Lockett, E. B. McKinney.

Entertainment: Henry J. Malochée, chairman, Thomas D. Miller, George A. Lederle, T. H. Sampson, V. L. Willoz.

Technical Papers: W. B. Wright, chairman, Arsène Perrilliat, Alex L. Black.

The postponed address of the past president, Mr. J. F. Coleman, was read by the Secretary, Mr. Coleman again being unavoidably absent. The address, " The Future Field for the Engineer in the State of Louisiana " was replete with suggestions as to the openings for the engineer. Railroads, irrigation, levees, drainage, sewerage, water works, electric-light systems and improvement of waterways were some of the many problems that would claim the engineer's attention. A new era was beginning for him in the state, one of prosperity and compensation worthy of his hire.

Mr. Coleman, in closing, thanked the Board of Direction and officers of the Society for assistance during his term of office.

The technical exercises of the evening, " The Panama Canal: A Discussion," were taken up. President Wright had prepared notes on the subject. He gave a brief history of the project from its incipiency to the present time. He then sketched what had been done by the United States since assuming charge of the Canal and what it was purposed doing.

A general discussion of the subject followed. The engineering features of the canal were discussed by Messrs. Wright, Duval, Raymond and others. The advantage, cost and expediency of a sea-level canal was gone into.

President Wright stated that at the next meeting " Public Improvements, Parks and Parkways " would be the subject of the evening's exercises. Mr. Allison Owen, president of the Central Parkway Commission, would be invited to address the meeting.

Adjourned.

G. W. LAWES, *Secretary.*

NEW ORLEANS, LA., APRIL 10, 1905. — The regular meeting of the Society was held at 8.30 P.M., President W. B. Wright in the chair. Ten members and two guests were present.

The minutes of the meeting March 13, 1905, were read and approved.

Mr. J. W. Armstrong, chairman of the Library Committee, reported that the committee had decided on binding the periodicals on hand, and purchasing a number of standard engineering works. He asked for suggestions from the members as to desirable books.

Mr. Alfred Théard, chairman of the Membership and Attendance

Committee, reported that the committee had formulated a plan that, it was hoped, would materially increase the membership. In the matter of attendance, if the coöperation of the Entertainment Committee could be secured, excellent results could be obtained. The plans were not ready to be submitted to the Society just yet.

Technical exercises were then taken up. President Wright had prepared notes on "Public Improvements, Parks and Parkways." He said that one of the first necessities of a city was provision for breathing spaces where the people could get fresh air and recreation. Existing conditions in New Orleans were explained, and the improvements that should be made pointed out. Now was the time to consider this question before land values became too high. The city was possessed of many boulevards that could be made very handsome, and the suburbs were susceptible of great improvement. Only one street in the city had been properly treated. Mr. Wright advocated a public square in the center of every five blocks radius.

Mr. Allison Owen, president of the Central Parkway Commission, who had been invited to address the meeting, was introduced.

Mr. Owen said parks and parkways in New Orleans had been neglected for the more important work of sewerage, drainage and water, but now the time had arrived when attention should be directed to civic embellishment. The boulevards of the city, referred to very admiringly by Park Engineer Olmstead in an address delivered in Portland, Ore., owed their width in nearly every instance to the fact that a drainage canal formerly occupied the center.

In selecting an avenue for a parkway from Audubon Park to City Park, Hagan Avenue had been looked upon as desirable because of its great width of 266 ft. The plans proposed for improving the avenue were explained.

Mr. Owen called attention to the unsightly conditions of the principal boulevards, due to telegraph poles, sign boards, etc., and cited the methods employed in Europe for abating this evil.

Mr. Owen advocated recreation piers on the river front, grouping of public buildings, etc. His remarks covered the subject very fully and were very much appreciated by the members.

The Chair announced that since the last meeting Mr. George A. Lederle, a member of the Society, had died. The following resolutions were then passed by the Society (for account of professional career see Obituary, page 117):

Resolved by the Louisiana Engineering Society, in regular meeting assembled:

That in the death of George A. Lederle the Society mourns the loss of a faithful, active and zealous member.

That the profession is deprived of the services of an untiring and energetic worker, a recognized authority on construction and an engineer of spotless character and reputation.

Be it further resolved, That the account of his professional life be filed with our records and that this and these resolutions be forwarded to his family as an expression of the respectful sympathy of this Society.

Adjourned.

G. W. Lawes, *Secretary.*

New Orleans, La., May 8, 1905. — The regular meeting of the Louisiana Engineering Society was called to order by President W. B. Wright. Twelve members and two guests were present.

The minutes of the meeting, April 10, 1905, were read and approved.

"Public Improvements, Parks and Parkways" was taken up for discussion.

The discussion was taken part in by President Wright, Mr. Allison Owen, present by invitation, Messrs. Coleman, Richardson, Armstrong, Miller and others, and dealt with the question of the requirements of New Orleans and the best means of securing results.

President Wright then introduced Prof. W. B. Gregory of Tulane University, who addressed the Society upon "Tests Made of a Pumping Plant" at Donaldsonville, La.

Professor Gregory said the purpose of the plant was to supply water from the Mississippi River, during the low stage, to five siphons which discharged into Bayou Lafourche over a dam at the head of the bayou. The object of the test was to determine if the pump came up to contract requirements. The contract called for a supply of 50 000 000 gallons of water every twenty-four hours. The test was made in the 36-in. discharge pipe at a point 150 ft. from the pump, the total length of the pipe being 530 ft. A Pitot tube was used, and four vertical traverses of the pipe were made, requiring one-half hour each. The test showed a rate of discharge of about 47 000 000 gallons per twenty-four hours, or 3 000 000 less than required.

The details of the test were interesting and the members enjoyed the address very much.

The Chair announced that at the next meeting Professor Gregory's address would be formally discussed.

Adjourned.

G. W. Lawes, *Secretary.*

New Orleans, La., June 12, 1905. — The regular meeting of the Society was called to order by President W. B. Wright. Ten members and two guests were present.

The minutes of the meeting, May 8, 1905, were read and approved.

Upon motion duly made and seconded, the Society decided to retain the present rooms another year.

The Society voted to adjourn, at the close of the meeting, until September.

An invitation from the Memphis Engineering Society to this Society, to be represented at its annual meeting, outing and banquet, was acted on, and Mr. Thomas Tutwiler, a member, resident in Memphis, was delegated to represent the Society.

The discussion of Professor Gregory's address, "Test of a Pumping Plant," was taken up.

Mr. Arsène Perrilliat, member of and representing the Board of State Engineers in the work of improving Bayou Lafourche, to which improvement the pumping plant is auxiliary, explained the necessity of the plant and detailed its installation. The failure of the plant to come up to requirements was due to improper impellers in the centrifugal and to excessive angles in the suction and discharge pipes. These de-

fects were to be remedied by the contractor, and it was thought would cure the trouble. The design of the pump was left to the contractor.

President Wright, Messrs. Haugh, Waddill, Raymond and others took part in the discussion.

Refreshments were served during the meeting.

Adjourned.

G. W. LAWES, *Secretary.*

NEW ORLEANS, LA., SEPTEMBER 11, 1905. — There was no quorum present and no meeting was held.

G. W. LAWES, *Secretary.*

NEW ORLEANS, LA., OCTOBER 9, 1905. — The regular meeting of the Louisiana Engineering Society was called to order by President W. B. Wright. Twelve members and four guests were present.

The minutes of the meeting, June 12, 1905, were read and approved.

The Library Committee reported the binding of twenty-eight volumes of periodicals.

The Chair announced that since the last meeting Mr. J. T. Norton, a member of the Society, had died. The following resolutions, prepared by President W. B. Wright and Secretary G. W. Lawes, a committee appointed for the purpose, were submitted to the Society and adopted (for account of professional life see Obituary, page 117):

Be it resolved by the Louisiana Engineering Society, in regular meeting assembled:

That in the death of John T. Norton the Society mourns the loss of a zealous member.

That the profession loses an active and energetic worker, a recognized authority upon railway construction and an engineer who possessed the respect and esteem of his fellow engineers.

Be it further resolved, That the account of his professional life be filed with our records, and that this and these resolutions be forwarded to his family as an expression of respectful sympathy of this Society.

Mr. A. M. Lockett then read a paper, " Design of Small Water-Works Pumping Plants: Some Practical Hints."

Mr. Lockett said the paper was intended to refer specially to small water-works pumping plants, the maximum capacity unit being 2 000 000 gal. in twenty-four hours. Engineers usually are inclined to attach too small importance to the pumping plant in water works, resulting in selection of improper type of pump and engine, injudicious selection of location and incomplete mechanical equipment.

The paper gave in detail the machinery, suction pipe, location of plant and type of pumping engine for different conditions. The manner of preparing specifications was explained. A comparison of the efficiency and economy of the different type of engines was made. The conclusion of the author was that no regular water-supply pump should be of lower grade than compound non-condensing.

Mr. Lockett advocated leaving the dimensions of the pump and engine to the manufacturer, limiting him to a safe piston speed.

Following Mr. Lockett's paper an informal discussion was had, in which Messrs. Miller, Malochée, Coleman, Armstrong and others took part.

The meeting then adjourned.

G. W. LAWES, *Secretary.*

Montana Society of Engineers.

BUTTE, MONT., OCTOBER 14, 1905. — The meeting of the Society for October was held in the Society room at the usual hour, with President E. W. King presiding. The minutes of the last meeting were read and approved. The Secretary reported the death of Mr. E. R. McNeill, a late member of the Society, and the President appointed a committee to draft resolutions on the death of Mr. McNeill; Messrs. Finlay McRae, Wm. F. Word and Frank L. Sizer constitute said committee. A vote of thanks was extended to Mr. Joseph H. Harper for his gift to the Society of twenty bound volumes of the Transactions of the American Institute of Mining Engineers.

The Society then adjourned.

CLINTON H. MOORE, *Secretary.*

Boston Society of Civil Engineers.

BOSTON, OCTOBER 18, 1905. — A regular meeting of the Boston Society of Civil Engineers was held at Chipman Hall, Tremont Temple, Boston, at 7.45 o'clock P.M.; Vice-President Otis F. Clapp in the chair; one hundred and twenty-five members and visitors present, including ladies.

The record of the last meeting was read and approved.

Mr. Howard S. Knowlton was elected a member and Mr. John J. Howard an associate of the Society.

Capt. William H. Jaques read the paper of the evening, entitled, "The Russian-Japanese War of 1904–1905; Its Scope and Meaning." The paper was illustrated by a large number of stereopticon views.

Adjourned.

S. E. TINKHAM, *Secretary.*

Engineers' Club of St. Louis.

ST. LOUIS, OCTOBER 4, 1905. — The 602d meeting of the Engineers' Club of St. Louis was held at the Club rooms, 3817 Olive Street, Wednesday evening, October 4, 1905. In the absence of President Flad, Mr. Greensfelder presided. Twenty-seven members and two guests were present.

The minutes of the 601st meeting were read and approved, and the minutes of the 393d meeting of the Executive Committee were read.

Upon motion of Mr. McCulloch the chair was authorized to appoint a committee to draw up a suitable memorial on the life of Mr. J. B. Guinn, a member of the Club who died recently.

The Secretary reported that Mr. Blaisdell whose death was reported at the last meeting was not a member of the Club at the time of his death.

The Secretary read a letter from Mr. W. R. Bascome, stating that Mr. Bascome had presented the Club with details and specifications of the new Manhattan Bridge in New York City.

The following were elected to membership in the Club: Raymond W. Dull, William B. Lemmon, Carl Oscar Nordensson, Ray E. Peshak, Kurt Toensfeldt, William C. Weidmann.

Mr. F. L. Douglas was proposed for membership.

Mr. H. J. Pfeifer presented a most interesting and instructive paper on " Street Paving in St. Louis." The discussion was spirited and valuable and was participated in by Messrs. Hanna, Purdon, Langsdorf, Robt. Moore, McCulloch, Russell, Fernald, Hasting, Pitzman, Wheeler, Lemmon, Parker and Bryan.

Mr. McCulloch gave some very interesting details of special street railway construction, with particular reference to the question of street paving.

Adjourned.

R. H. Fernald, *Secretary.*

St. Louis, October 18, 1905. — The 603d meeting of the Engineers' Club of St. Louis was held in the Club rooms, 3817 Olive Street, Wednesday evening, October 18, 1905, President Flad presiding. Present also were nineteen members and three guests.

The Chair appointed A. S. Langsdorf secretary *pro tem.*, in the absence of the regular secretary, R. H. Fernald.

The minutes of the 602d meeting of the Club were read and approved. The minutes of the 394th meeting of the Executive Committee were read.

Professor Van Ornum called attention to the fact that in the Proceedings of the American Society of Civil Engineers the location of the Club rooms is incorrectly stated, no change in the announcement having been made since the removal to the new quarters. The Secretary was requested to notify the secretary of the American Society of Civil Engineers of the change.

The Chair announced the appointment of Mr. A. P. Greensfelder to the chairmanship of the Entertainment Committee vice Mr. W. G. Brenneke, resigned.

The paper of the evening, on " Azimuth from Polaris and a Southern Star," by Dr. George O. James, of Washington University, was then read in abstract by the author. Dr. James described a method, developed by himself, which is capable of locating a meridian within one-tenth minute of arc, and which requires but one setting of the transit and an approximate knowledge of the longitude and sidereal time of the place where the observation is made. The necessary calculations are very short and simple. Dr. James stated that trials of the method by unskilled observers (students of his) gave results which differed from each other and from the mean by only a small fraction of one per cent.

The paper was discussed by Professor Van Ornum, Dr. James and Mr. Bouton.

Adjourned.

A. S. Langsdorf, *Secretary pro tem.*

Association of Engineering Societies.

Vol. XXXV. OCTOBER, 1905. No. 4.

PROCEEDINGS.

Engineers' Club of St. Louis.

St. Louis, November 1, 1905. — The Engineers' Club of St. Louis held its 604th meeting at the Club rooms, 3817 Olive Street, Wednesday evening, November 1, 1905. In the absence of the President, Vice-President Layman presided. Thirty-four members and eighteen guests were present.

The minutes of the 603d meeting were read and approved. The minutes of the 395th meeting of the Executive Committee were read.

The Secretary read a letter from Mr. C. W. Hunt, Secretary of the American Society of Civil Engineers, extending the privileges of the rooms of the American Society of Civil Engineers in New York to the members of the Engineers' Club of St. Louis.

A letter was also read from Mr. Pfeifer of the Terminal Railway Association, offering a special train for the proposed trip to Madison, Ill., Saturday, November 4.

The Secretary reported that he had taken proper action regarding these matters.

Mr. F. L. Douglas was elected to membership in the Club.

The plan outlined by the Executive Committee for a Bureau of Information was unanimously approved by the thirty-four members of the Club present.

The following names were proposed to serve on the Nominating Committee to nominate officers for 1906: Messrs. McCulloch, Fay, Childs, Pfeifer, Kinealy, Dziatzko, Schmitz, Bouton and Laird.

The five following were elected: Messrs. McCulloch, Fay, Childs, Pfeifer and Laird.

Mr. W. A. Layman's paper upon the " Progress in Electrical Power Transmission " was of unusual interest and value. It was profusely illustrated by means of slides and was discussed by Messrs. Van Ornum, Helm, Langsdorf, Humphrey and Marshall.

Adjourned.

R. H. Fernald, *Secretary.*

Association of Engineering Societies.

Vol. XXXV. **NOVEMBER, 1905.** No. 5.

PROCEEDINGS.

Civil Engineers' Club of Cleveland.

Regular Meeting, November 14, 1905, at Club rooms, called to order by the President, at 8.15 p.m. Present, sixty members and visitors.

Minutes of preceding meeting read and approved.

Applications from the following for active membership, approved by the Executive Board, were read: Louis A. Corlett, Philo A. Orton, M.S., George H. Rose and Adolphus E. Sprackling.

Communications from the following were read: Mrs. C. H. Wellman, Chas. Warren Hunt, secretary of the American Society of Civil Engineers, inviting the members of this Club to make use of the facilities of the society when in New York, and from Mr. R. R. Richardson, superintendent of the Pittsburg and Conneaut Dock Company.

A report of the Committee on Legislation relative to water pollution, hereto appended, was read and ordered received and filed.

The following resolution by Mr. Rice was unanimously adopted after a somewhat extended discussion:

Resolved, That a standing committee of three members be appointed, with power to call a meeting of delegates from engineers' clubs and other organizations in Ohio with a view to determining on proper action to be taken relative to the subject of water pollution, and that the sum of twenty-five dollars ($25.00) be hereby appropriated from the general fund of the Club to defray necessary expenses of this committee.

The President afterward named as members of this committee the same members who had presented the report above referred to, viz., Colonel Kingman, Mr. Rice and Mr. Beardsley.

The Secretary announced the presentation to the Club, by Mr. William H. Hunt, past president of The Builders' Exchange, of a handsomely bound set of the official photographs, taken for the British Government, of the Assouan Dam, Egypt. The Secretary was instructed to convey to Mr. Hunt the Club's appreciation of his handsome gift and its thanks for it.

The reading of the paper of the evening, " Fifty Years' Record of the Weather at Cleveland, Ohio," by Mr. G. A. Hyde, a charter member of the Club and the guest of honor, then followed. The paper and Mr.

Hyde's remarkable achievement in completing fifty years of unbroken weather observations were then interestingly commented upon by Mr. James Kenealey, local U. S. observer, Father Odenbach of St. Ignatius College and Ensign W. L. Varnum, U. S. Navy, local hydrographic officer, who were the especial guests of the evening; and by Mr. Paulin in a carefully prepared paper, Mr. Warner, Mr. McGeorge, Mr. Dutton and others.

An enjoyable social session with refreshments followed adjournment.

Adjourned.

Jos. C. Beardsley, *Secretary.*

REPORT OF COMMITTEE ON WATER SUPPLY POLLUTION

October, 1905.

The Civil Engineers' Club of Cleveland:

Your committee to formulate legislation relative to pollution of sources of water supply for municipalities in Ohio beg to report as follows:

First. We believe that the magnitude of this undertaking is such that this committee cannot undertake, unaided and without much more information than it now has, to formulate any legislation to be introduced at the coming session of the state legislature.

Second. The ultimate action to be taken should not be confined to the state of Ohio, but should probably include all the states on the Great Lakes and possibly some others.

Third. Two ways have suggested themselves to your committee in which such action might be brought about, one of which is the calling of a convention of all technical organizations and such others as might be interested in the subject, in the territory above indicated, this convention to endeavor to agree on proper legislation to be introduced simultaneously in the legislatures of all the states concerned. The other is to secure the coöperation of all such bodies in Ohio to the end that legislation be secured in our own legislature providing for a paid commission of experts to make an exhaustive study of the problem with a view to obtaining proper simultaneous action of all the states.

Fourth. In view of the expense involved in initiating either of the projects mentioned above, it is recommended that any committee intrusted with carrying them out be given an appropriation of not less than $25.00.

Fifth. Your committee believes that the problem sought to be solved is one of vital importance to all municipalities and to all engineers who have to do with water supply or sanitary works.

Respectfully submitted,

D. C. Kingman,
W. P. Rice,
Jos. C. Beardsley,
Committee.

Boston Society of Civil Engineers.

Boston, Mass., November 15, 1905. — A regular meeting of the Boston Society of Civil Engineers was held at Chipman Hall, Tremont Temple, Boston, at 8 o'clock p.m., President John W. Ellis in the chair. Sixty-three members and visitors present.

The record of the last meeting was read and approved.

Messrs. Bertram E. Ames, William W. Burnham, George W. Cutting, Jr., and Charles Saville were elected members of the Society.

The President announced the death of Frank L. Fales, a member of the Society, which occurred October 5, 1905, and on motion of Mr. R. A. Hale the President was requested to appoint a committee to prepare a memoir. The President has appointed as that committee, Prof. L. J. Johnson.

Mr. George B. Francis read the paper of the evening, entitled "Construction of Water Power on the Chattahoochee River at Atlanta, Ga." The paper was illustrated with lantern slides.

Adjourned.

S. E. Tinkham, *Secretary.*

ASSOCIATION
OF
ENGINEERING SOCIETIES.

VOL. XXXV. DECEMBER, 1905. NO. 6.

PROCEEDINGS.

Montana Society of Engineers.

THE regular meeting of the Society for November was held at the usual place at 8 P.M., with Vice-President Dunshee presiding. The minutes of the October meeting were approved. The report of the Committee on Nominations of Officers for next year was read and approved, and the Secretary was instructed to prepare the ballots to be mailed to the members in December. The candidates named are as follows:

President — Bertram H. Dunshee, of Butte.
First Vice-President — Edward C. Kinney, of Bozeman.
Second Vice-President — Archer E. Wheeler, of Great Falls.
Secretary and Librarian — Clinton H. Moore, of Butte.
Treasurer and Member of the Board of Managers of the Association of Engineering Societies — Sam'l Barker, Jr., of Butte.
Trustee — Geo. W. Wilson, of Butte.

The chair named the following committee to recommend a place for holding the annual meeting: Messrs. King, Goodale and Bowman. The committee will report at the December meeting. The Committee on Mining Laws reported progress. The trustees recommended the publication in the Association Journal of President Moulthrop's annual address and the papers of Mr. Chas. W. Goodale and Messrs. Carroll and Starz, and the Secretary was instructed to forward the manuscripts for publication.

The Secretary called the attention of the members present to the very great need of additional bookcases for new publications on hand and rapidly accumulating.

After a short discussion of a recent magazine article on the Panama Canal project, the Society adjourned.

CLINTON H. MOORE, *Secretary.*

Engineers' Club of St. Louis.

St. Louis, November 15, 1905. — The Engineers' Club of St. Louis held its 605th meeting at the Club rooms, 3817 Olive Street, Wednesday evening, November 15, 1905, President Flad presiding. Forty-four members and nineteen guests were present.

The minutes of the 604th meeting were read and approved. The report of the 396th meeting of the Executive Committee was read.

Mr. F. H. Pearson's application for membership was read and referred to the Executive Committee.

The Secretary read letters from Mr. Philip Moore and from Mr. J. V. Hanna relating to vacancies for which they desire men.

The Nominating Committee appointed at the last meeting reported as follows:

St. Louis, November 10, 1905.

The Nominating Committee appointed at the meeting of November 1, 1905, respectfully submit the following nominations for officers of the Engineers' Club of St. Louis, for the year 1906:

President, W. A. Layman; Vice-President, E. R. Fish; Secretary, R. H. Fernald; Treasurer, E. E. Wall; members of the Executive Committee, C. A. Moreno, C. D. Purdon; members of the Board of Managers of the Journal, A. P. Greensfelder, Hans C. Toensfeldt.

Respectfully submitted,

(Signed) Richard McCulloch, *Chairman.*
John A. Laird.
E. B. Fay.
H. J. Pfeifer.
O. W. Childs.

Upon motion of Mr. Fay, a vote of thanks was extended to Prof. J. H. Kinealy for a copy of his book on Centrifugal Fans recently presented to the Library.

Upon motion of Mr. Greensfelder a vote of thanks was extended to the following gentlemen who assisted in making enjoyable the recent trip of the Club to the works of the American Car and Foundry Company, in Madison, Ill.:

Mr. W. H. McBride, First Vice-President, Mr. Ames, Chiet Engineer, American Car and Foundry Company; Mr. H.J. Pfeifer, Engineer Maintenance of Way, Terminal Railroad Association; Mr. W. B. More, Superintendent Frisco Railroad.

It was moved by Mr. Fay that the Entertainment Committee be authorized to look after the arrangements for the annual dinner of the Club. Motion was carried.

The paper of the evening, upon the Recent Development of the Gas Producer and Large Gas Engine, was presented by R. H. Fernald.

After liberal discussion by Messrs. P. Moore, Robert Moore, Kurt Toensfeldt, Russell, Weidman, Kinealy, Reeves, Quam, McCulloch, Wheeler, Phillips and Fernald, the meeting adjourned.

R. H. Fernald, *Secretary.*

Montana Society of Engineers.

Butte, Mont., December 9, 1905. — The regular meeting of the Society was held December 9 at the Society Room at the usual hour, 8 p.m., President E. W. King presiding. The minutes of the November meeting were approved as read. The Secretary presented the applications for active membership of Howard Irwin Shaw and John Dawson Pope, and one from George Mittenberger for junior membership. The applications were approved and the Secretary instructed to send out the necessary ballots. The Committee on Annual Meeting reported in favor of holding same at Lewistown, Mont., and the report was adopted. The Chair appointed the following Committee of Arrangements: Messrs. McClean, McArthur, King, Goodale and Bowman; also a Transportation Committee consisting of Messrs. Carroll, Moore and Wheeler.

The committee appointed to draft resolutions on the death of E. R. McNeill submitted the following, which were approved:

Whereas, God in his providence has removed from our midst Brother E. R. McNeill, a member of this Society, now, therefore, be it

Resolved, That in the death of Brother McNeill this Society has suffered an irreparable loss. His sterling qualities of head and heart were well known to his intimate friends, and his conscientious discharge of every duty intrusted to him is testified to by his employers as well as by those associated with him on engineering work.

During a long period of active service on railroad engineering in Montana Mr. McNeill was known as one of the most thorough and painstaking engineers, on some of the most difficult work ever executed in Montana.

Resolved, That this Society shall express by these resolutions its sincere sorrow on the death of Mr. McNeill, and these resolutions shall be spread upon the minutes of the Society and a copy forwarded to his bereaved family.

Finlay McRae,
Wm. F. Word,
Frank L. Sizer,
Committee.

The Secretary presented the resignation of Trustee Edwin L. Blossom and the same was accepted with regrets. After an informal discussion of plans for the annual meeting the Society adjourned.

Clinton H. Moore, *Secretary.*

C. A. V. N.